CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

DECEMBER, 1953. **Vol. XLVIII, No. 12**

FORTY-EIGHTH YEAR OF PUBLICATION

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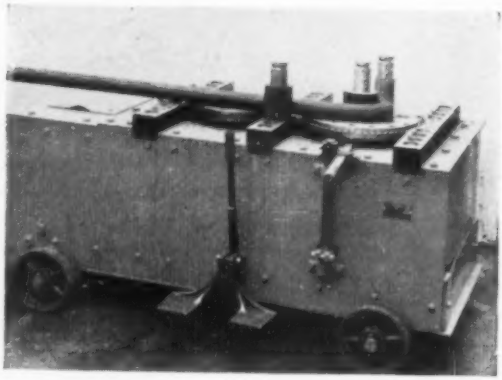

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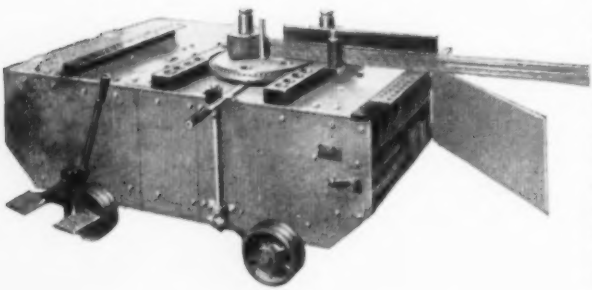

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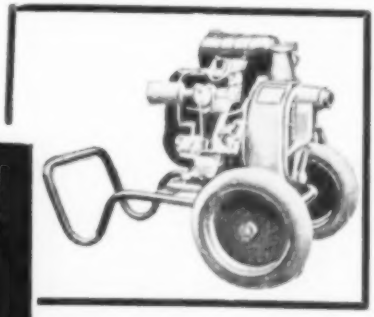

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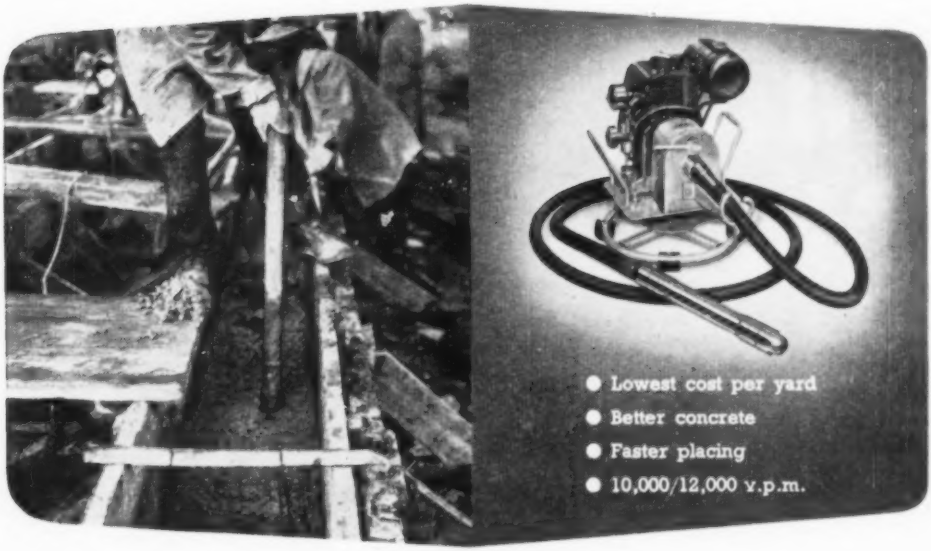

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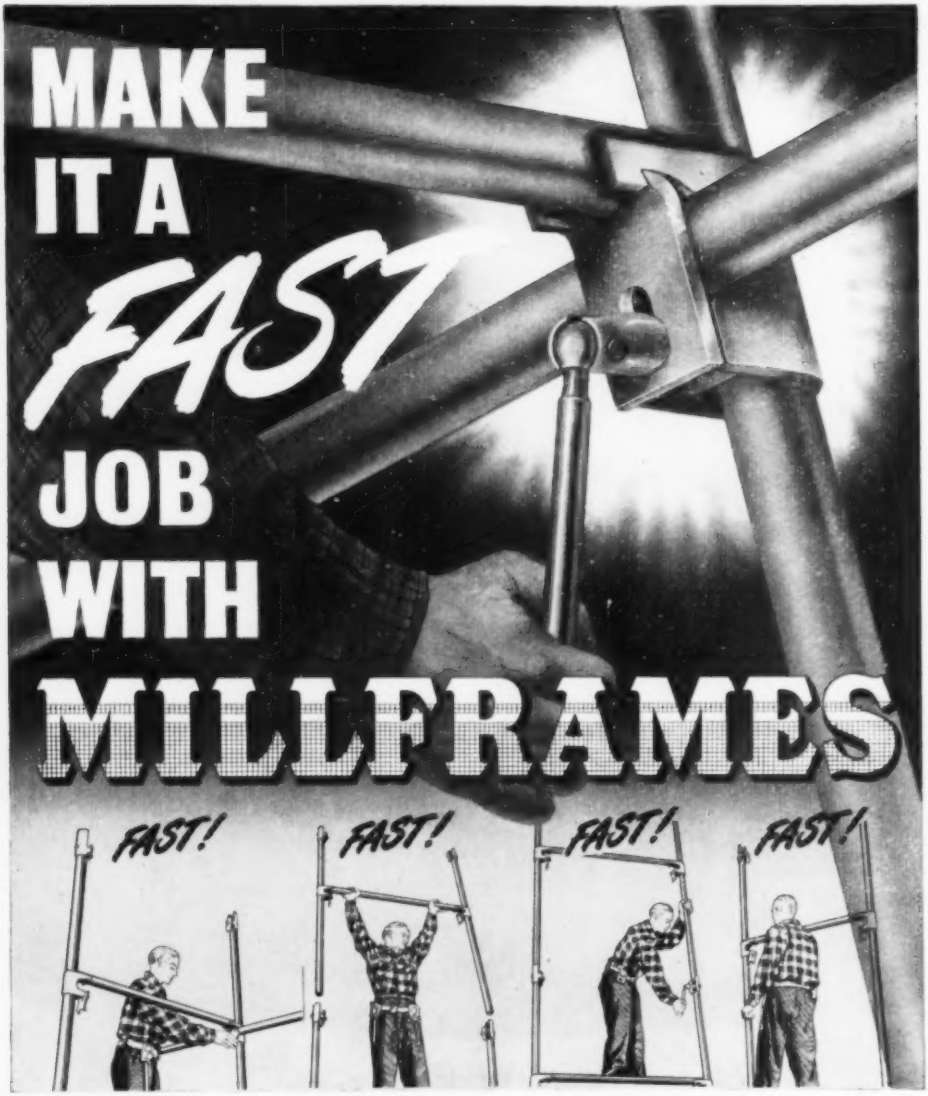

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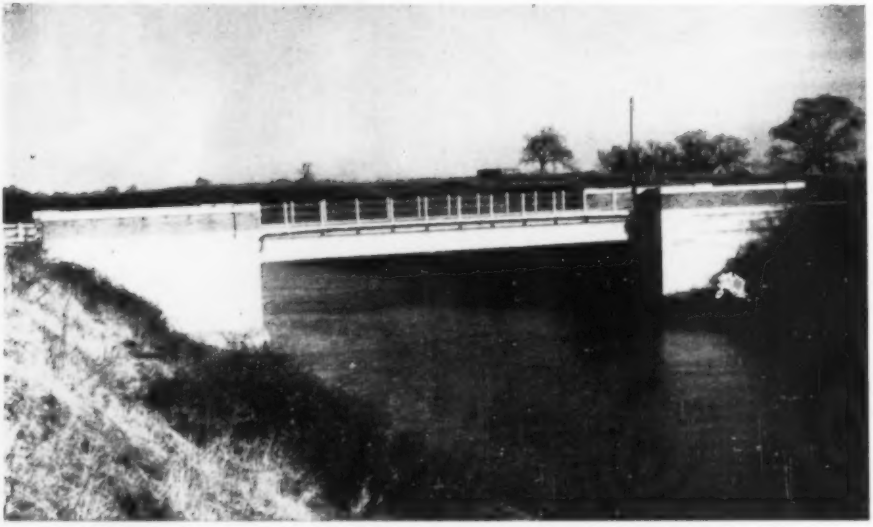

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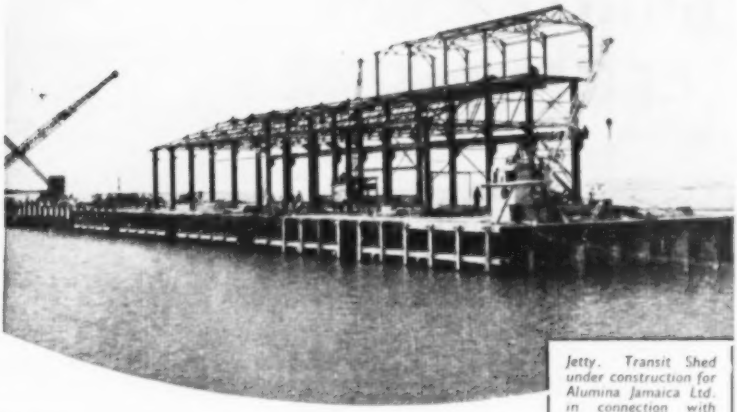

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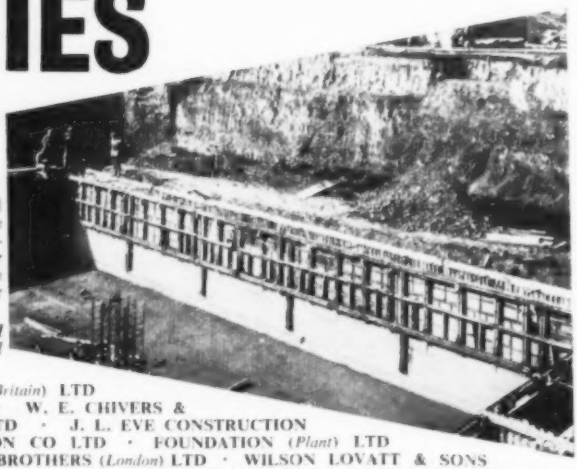

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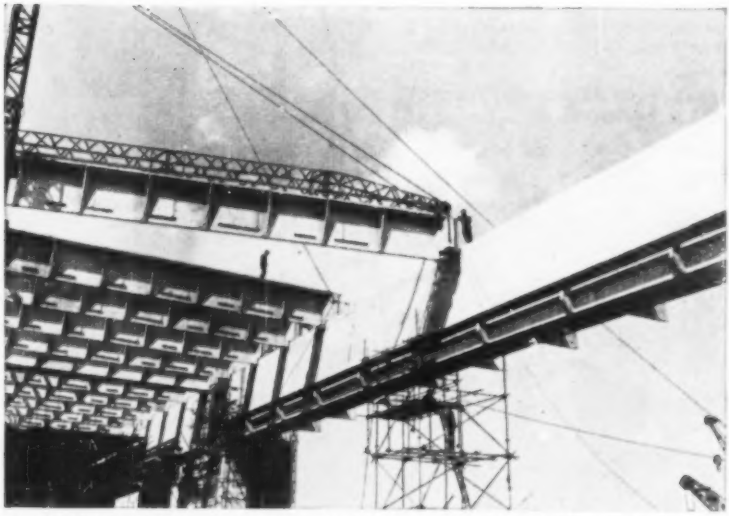

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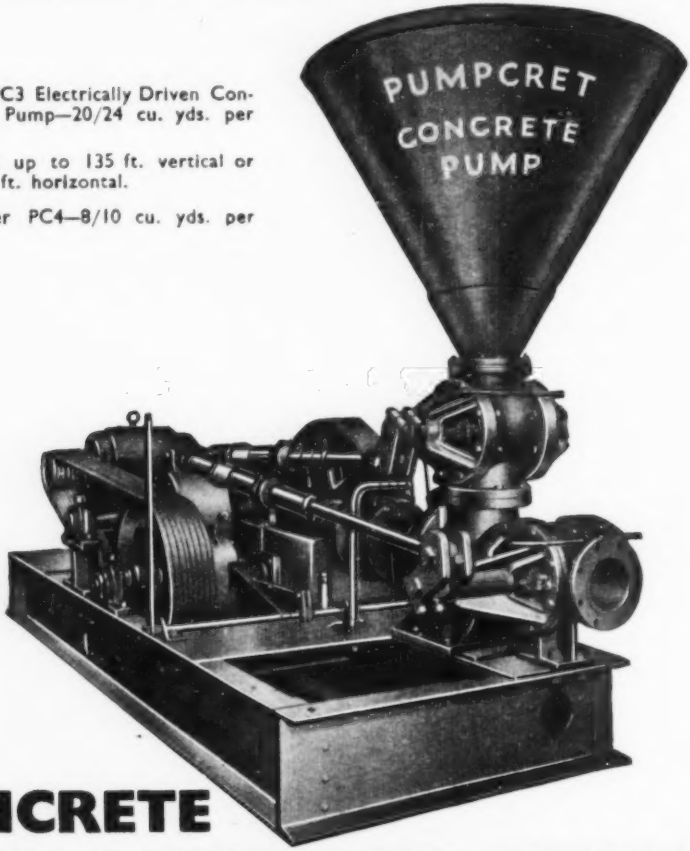

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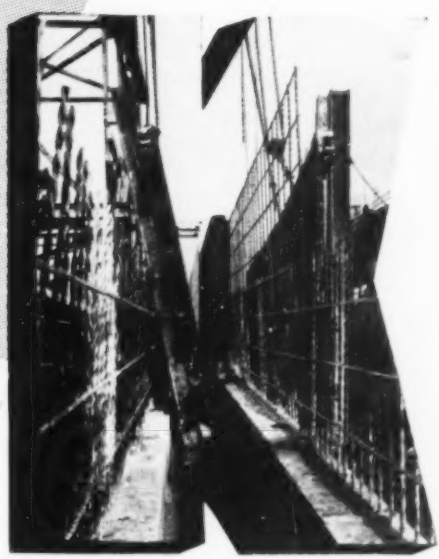

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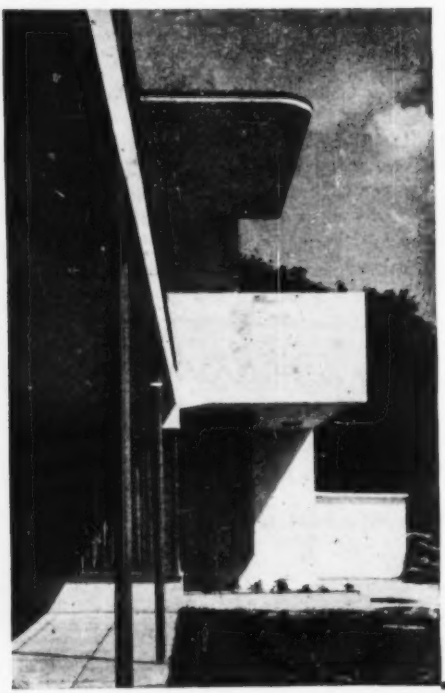

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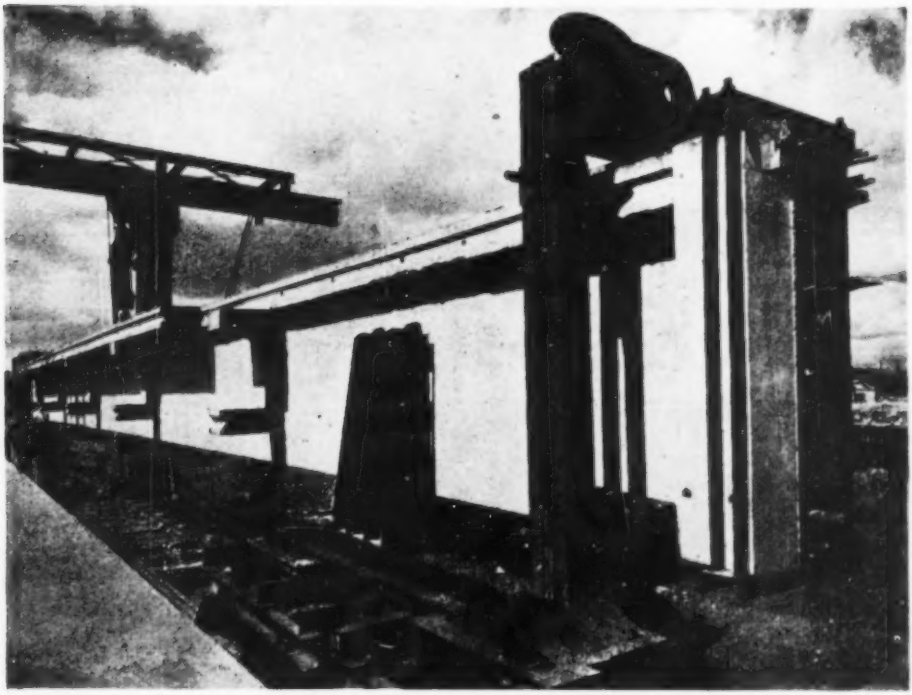

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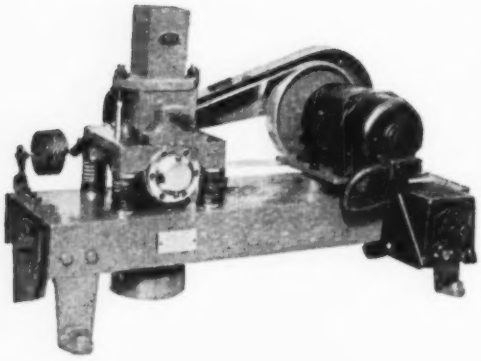

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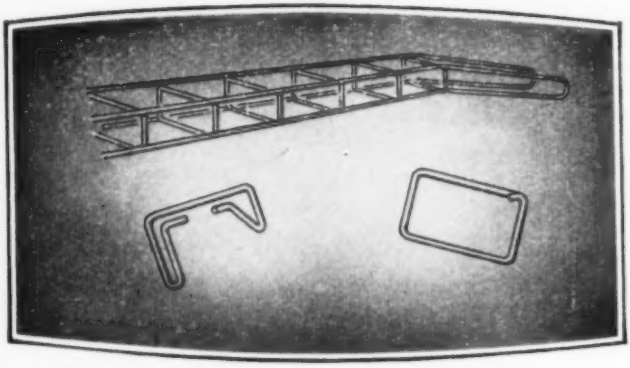

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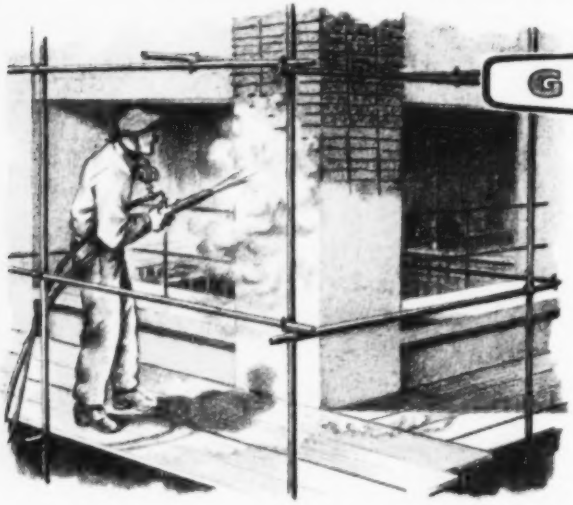

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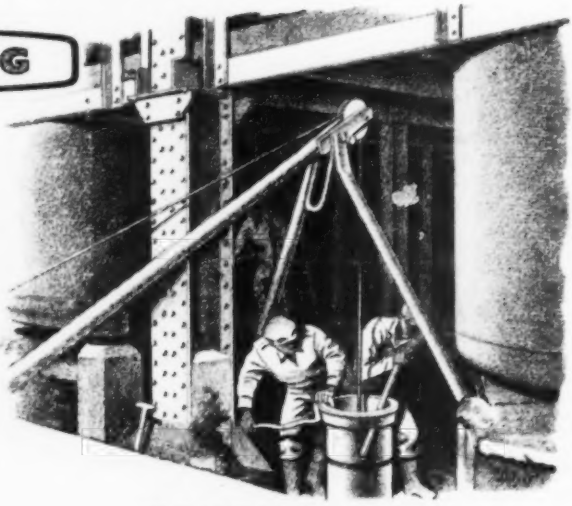

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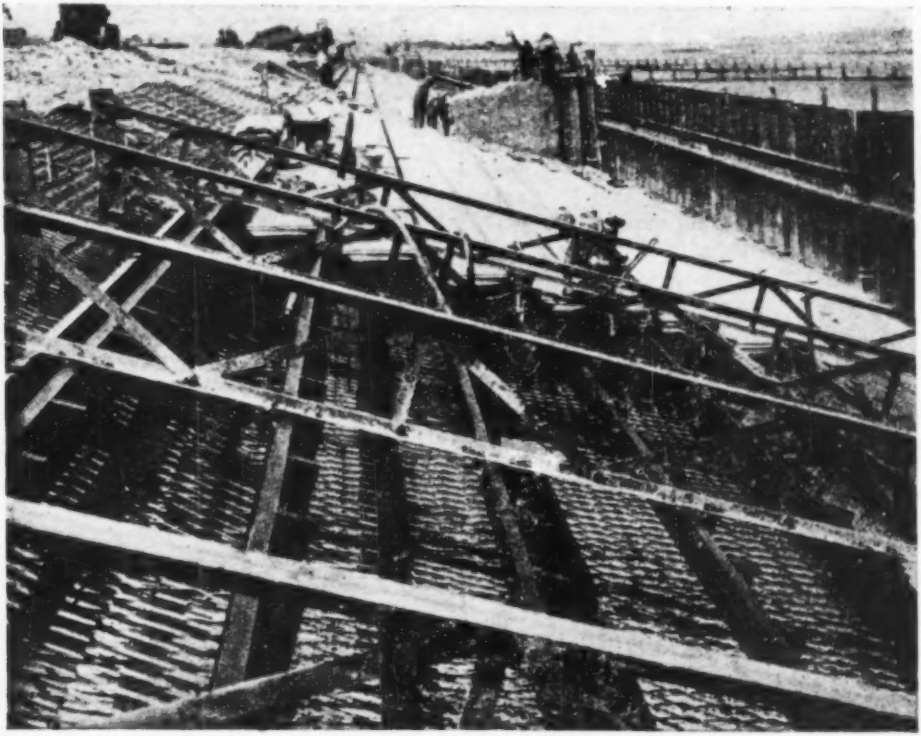

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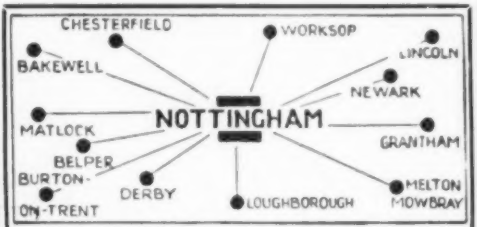

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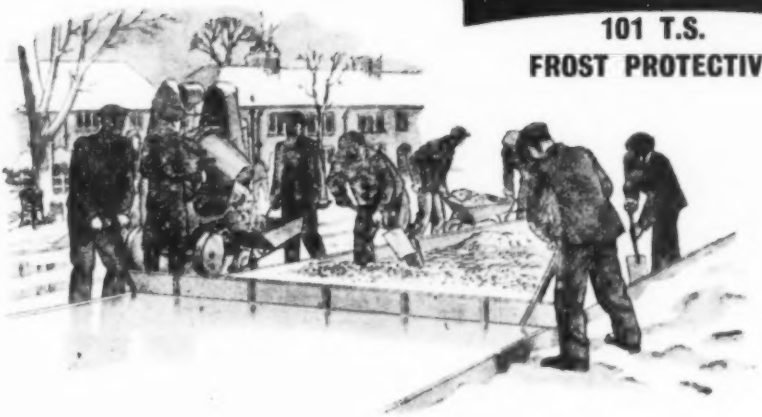

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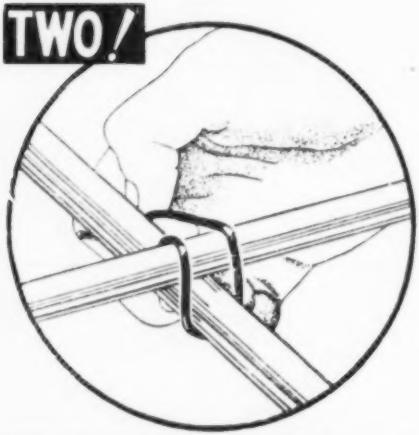

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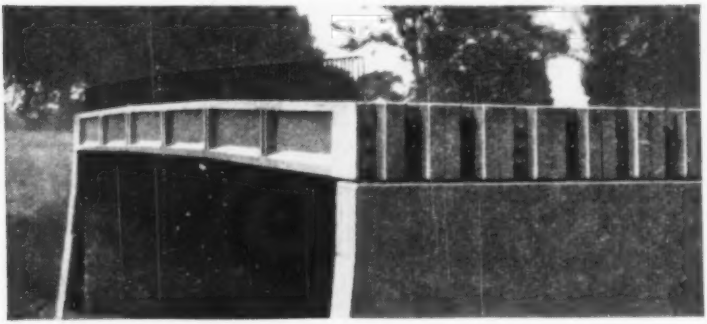

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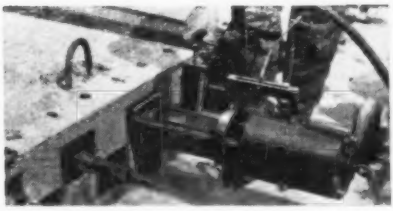

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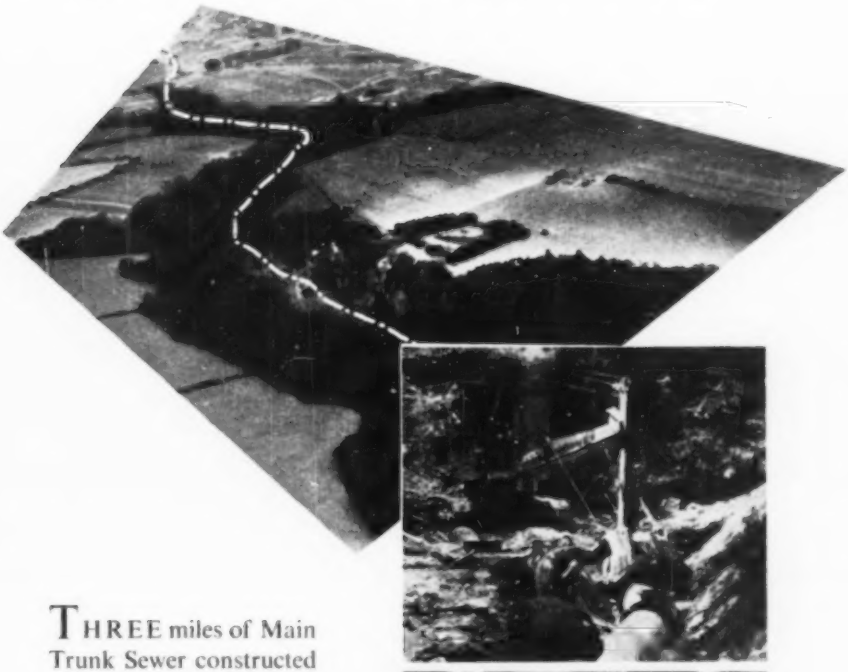

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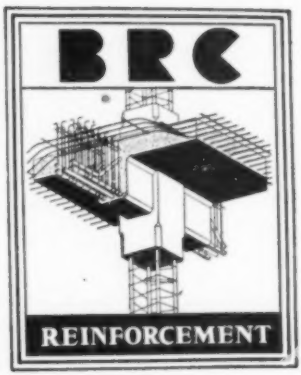

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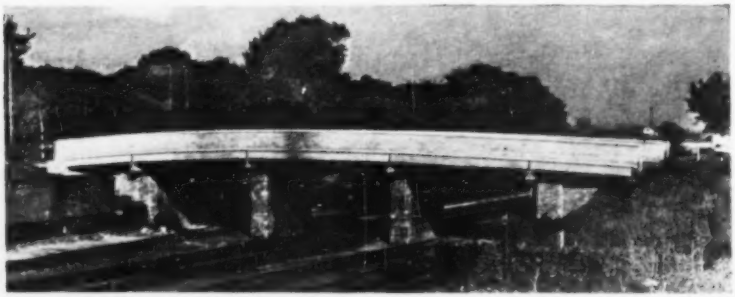

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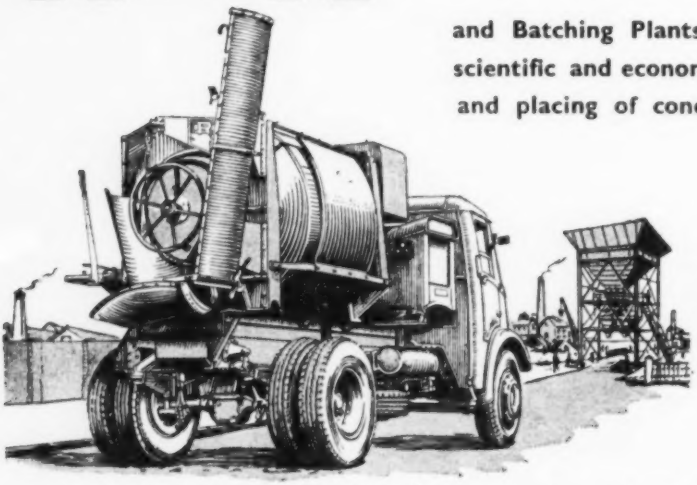

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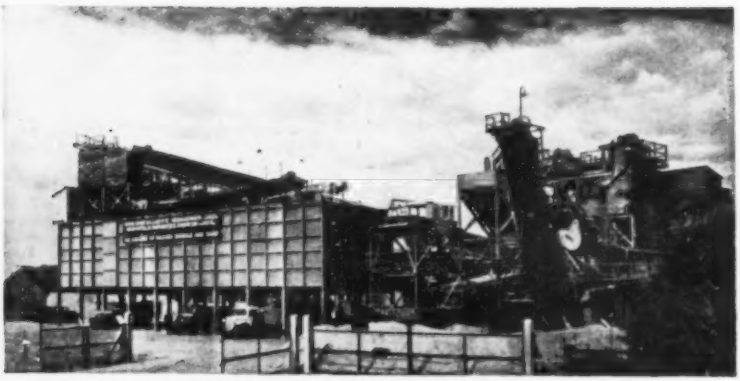

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Volume XLVIII, No. 12. LONDON, DECEMBER, 1953

EDITORIAL NOTES

Broader Minds.

WE have in this journal for many years deplored the lack of general education and initiative of many of the young people who have spent their most formative years in technical training at the universities and technical schools. This problem has been discussed recently in both Houses of Parliament and elsewhere, and last month it was mentioned by the Duke of Edinburgh whose remarks were the subject of a leading article entitled "Broader Minds" in "The Times" for November 5. This article deserves the study of those responsible for technical training, and of the students themselves, and we therefore, with permission, reprint it in full.

"The Duke of Edinburgh chose as one of his main themes at Edinburgh University yesterday the evils of that narrowness of mind which too many of the products of good schools and universities in these days seem to show. It is a general charge. University teachers, confronted with their freshmen, bewail their minds prematurely bowed down with learning in one single field. Employers in their turn exclaim against graduates who cannot speak, cannot write, certainly cannot manage men, and are fit only to spend their days in a laboratory corner. Some caution needs to be shown. Because nearly twice as many people are sent to universities, it does not follow that twice as many talented graduates have been produced. In fact, one of the lessons of expansion has been that if more men of the middling sort have presented themselves, the number of first-class men has shown little sign of increase.

"At the same time, this is no reason why the graduate of middling abilities, who is much needed, should be so sunk in his chosen academic groove as he often is. Mr. Robert Birley, at the British Association meeting this year, talked, as many have done before him, of the 'great divide' between humanists and scientists, of the loss of understanding between Civil servants and research workers. The need is for a common foundation of knowledge which both may share, and the Duke of Edinburgh made a useful distinction when he said that school was the place where that foundation should be laid. There is a tendency nowadays to argue that universities should institute compulsory general courses to curb the narrowing influence of specialist studies. Universities, however, are the right place for specialization. 'It is in school and not at the university,' as the Duke of Edinburgh said, 'that the budding scientist should be helped, for instance, to develop a taste for music and the arts, and the young historian an understanding and reverence for science.'

" It is not only, of course, a taste for music and the arts that the young scientist and young engineer need. If they wish to climb from specialized duties to the general task of managing men—and it is a healthy aim to have more technical experts on directing boards—they will need to have studied men in literature and history. If these broadening influences are unfelt in the sixth form it is owed entirely to the specializing bent of English education, encouraged by university entrance and scholarship requirements. No other country in the world specializes as the English do in their grammar schools between the years of fifteen and eighteen. Where the Germans, say, will give one-third of the week to specialist subjects, an English grammar school will give two-thirds. As the Duke of Edinburgh said : ' The schools cannot concentrate on the broad training of the intellect and the building of character if the universities are going to demand narrow academic qualifications for their entry requirements.' Certainly the universities must insist upon a high standard in their entrants, on work well done ; they will not want to encourage a welter of general cultural nonsense instead of sound learning. But they need to give most careful thought to what they can do, through their entrance requirements, to encourage a broader approach in the sixth forms of schools."

The foregoing does not exaggerate a common result of the present method of preparing a young man for a scientific or technical career. This generally takes the form of a scanty general education followed by a period of technical training during which the students' minds are confined almost entirely to a narrow line of thought which seems often to produce men adept at little more than computing stresses. Indeed in some branches of engineering it seems to be the practice to describe such people as " stressmen."

It is not, however, our view that the remedy lies entirely with the schools. It is very desirable that less time should be devoted to technical subjects at schools, particularly in view of the difficulty of obtaining qualified teachers, but it is also important that a liberal education should not end when a student enters a university or a technical school. Liberal teaching at the schools would be largely wasted if it ended when the student ceases to be a schoolboy. Generally the technical training calls for little independent thought. Lecture notes are made and remembered (or not). The student is told, or finds out, that the answers to his problems are to be found in published literature ; he is taught which formulæ are applicable to various problems, and uses them. If he is to pass his examinations the student will generally find that a good memory and a knowledge of mathematics are more important than imagination and independent seeking for the truth. It is our view that before he begins to specialise the technical student's first year or so at the university should be devoted to a broader education, which will enable him more easily to deal with scientific and technical problems. It is indeed tragic that so many men should leave a university with a sound knowledge of the subject they have studied but without the ability to express themselves in plain English and with false ideas of the meanings of words ; that they should be only " stressmen " or, as the writer in " The Times " suggests, " fit only to spend their days in a laboratory corner ", is a waste of expensive educational facilities at a time when in so many directions Great Britain is lagging behind the rest of the world in inventiveness.

Loads on Groups of Free-standing Piles.

By C. E. REYNOLDS, B.Sc. (Eng.), A.M.I.C.E.

THE following notes [1] relate to the calculation of the load on each pile in a group of piles which project some distance above the ground, that is on free-standing piles such as occur in wharves and jetties. It is assumed that the external forces on the group of piles can be defined by the position and magnitude of a horizontal force H and a vertical force W, the force H being primarily due to inward blows from ships berthing or to the outward pull on mooring ropes. The vertical force is due to loads from cranes, vehicles, and the like, the weight of stored goods, and the weight of the structure. The effect of probable combinations of various values of H and W should be investigated to determine the most adverse effect on the piles. It is also assumed that the piles are surmounted by a rigid cap (or, if all the piles are vertical, that they are rigidly braced transversely at a height h above the horizontal datum) so that the deformation of the superstructure is assumed to be negligible compared with the deformation of the piles below.

Structures with Vertical Piles Only.

A structure on vertical piles only is shown in *Fig.* 1, which also gives the notation used in the formulæ. The loads on the piles are determined by the simple statical analysis given in Appendix I, involving W and a moment depending

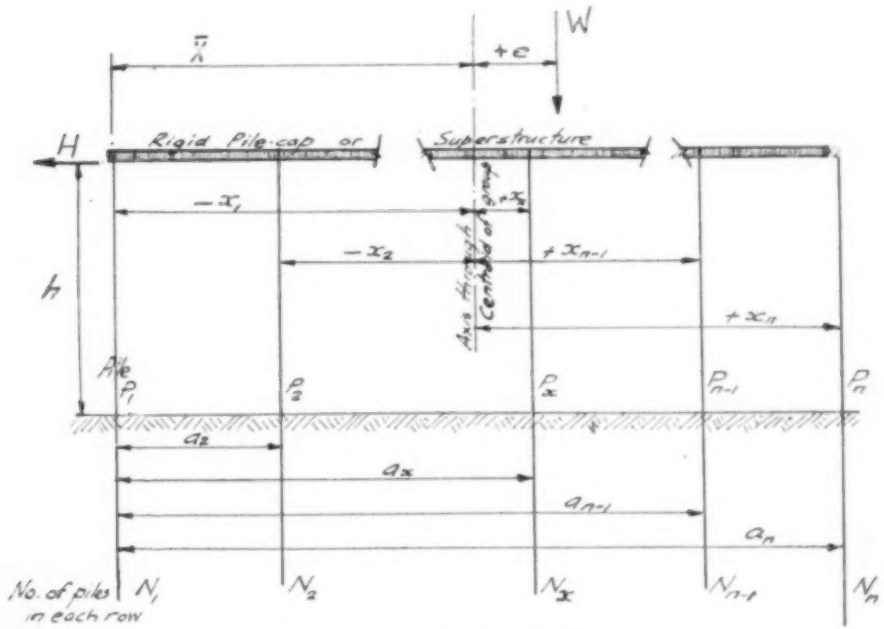

Fig. 1.—Group of Vertical Piles.

[1] The data given in this article are the basis of *Table No.* 19 in the writer's "Reinforced Concrete Designer's Handbook," in which some examples of the application of the data are given.

on H, h, and the eccentricity of W. The datum from which h is measured is the level at which the piles are rigidly embedded in the ground; with most types of soil this is assumed to be the level of the ground or within, say, 5 ft. below ground level. The horizontal force is transferred to the ground by shearing forces which produce bending moments on the piles. Since the piles offer relatively little resistance to bending, structures with vertical piles only are not suitable when the horizontal force is large. The present analysis assumes that the datum from which h is measured is horizontal and the shearing force on each pile is equal; therefore the bending moment on each pile is the same. Modifications must be introduced if the plane through the points of embedment in the ground is inclined, the basic principle being that the horizontal displacement of the head of each pile in relation to the point of embedment is the same for each pile.

For the conditions assumed in *Fig.* 1, the total axial load P_x, the shearing force S_x, and the bending moment M_x on any pile in a group of vertical piles is, as shown in Appendix I, given by

$$P_x = \frac{W}{\Sigma N} + \frac{x(We - Hh)}{N_x \Sigma N x^2} + W_x \quad . \quad . \quad . \quad . \quad (1)$$

$$S_x = \frac{H}{\Sigma N} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$M_x = \frac{hH}{2\Sigma N} = \frac{S_x h}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

In formulæ (1), (2), and (3), W, H, x, and e are assumed to be positive (*Fig.* 1); the sign in front of any of these terms which are not positive must be changed. ΣN is the number of piles in the group; W_x is the weight of a pile; e is the distance from the centroid of the group to the line of action of W.

Structures with Inclined and Vertical Piles.

When the horizontal load predominates, inclined piles should be provided either entirely or in addition to vertical piles because the resistance to a horizontal force of an inclined pile is many times greater than that of a vertical pile. Therefore, in a group containing inclined and vertical piles, the horizontal forces are resisted by the inclined piles, the resistance of the vertical piles being neglected in this respect. In such groups the bending moments and shearing forces on the piles are negligible. It is generally assumed that (i) each pile is hinged at the head and toe, and (ii) the pile-cap is rigid. These assumptions, especially (i), are not accurate, but the results obtained by theories based thereon differ little from the results of analyses [1] based on fixity at the ends of the piles. The restraining effect of the ground into which the piles are driven is considered by Mr. S. Jampel in an article entitled "An Analysis of Groups of Piles" in this journal for July and August, 1949.

In the following is described a practical adaptation of the common theory given in Appendix II. The notation and signs are given in *Fig.* 2. The constant Q_x for the properties of any pile is $\frac{EA_x}{L_x}$, E being the modulus of elasticity. The

[1] Such an analysis is that of Nökkentved in "Berechnung von Phahlrosten," quoted by Professor G. Magne in "Stabilité des Constructions," Vol. II.

vertical component V_x of the axial reaction of any inclined pile the head of which is subjected to unit vertical depression is $Q_x \cos^2 \theta_x$. The point O, at which intersect the line of action of the resultant of the axial reactions of the piles due to unit vertical depression downwards and the line of action of the resultant of the axial reactions due to unit horizontal movement towards the left, can be considered as the centre of the system. If the origin of the system of co-ordinates is the point where the left-hand pile of the group meets the pile-cap, the co-ordinates x_0 and y_0 of O are given by

$$\left.\begin{aligned} x_0 &= \frac{\beta\dfrac{\Sigma V_x x_x}{\Sigma V_x} - \alpha\dfrac{\Sigma V_x x_x \tan\theta_x}{\Sigma V_x \tan\theta_x}}{\beta - \alpha} \\ y_0 &= \frac{\dfrac{\Sigma V_x x_x}{\Sigma V_x} - \dfrac{\Sigma V_x x_x \tan\theta_x}{\Sigma V_x \tan\theta_x}}{\beta - \alpha} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where α and β are $\dfrac{\Sigma V_x \tan\theta_x}{\Sigma V_x}$ and $\dfrac{\Sigma V_x \tan^2\theta_x}{\Sigma V_x \tan\theta_x}$ respectively.

It is also shown in Appendix II that if a planar group of piles is subjected to a vertical force W (acting downwards) and a horizontal force H (acting towards the left) the load P_x on any pile, excluding the weight of the pile, is given by

$$P_x = \frac{V_x}{\cos\theta_x}\left[\frac{W(\beta - \tan\theta_x)}{(\beta - \alpha)\Sigma V_x} + \frac{H(\tan\theta_x - \alpha)}{(\beta - \alpha)\Sigma V_x \tan\theta_x} + \frac{MX_x}{I}\right] \quad . \quad (5)$$

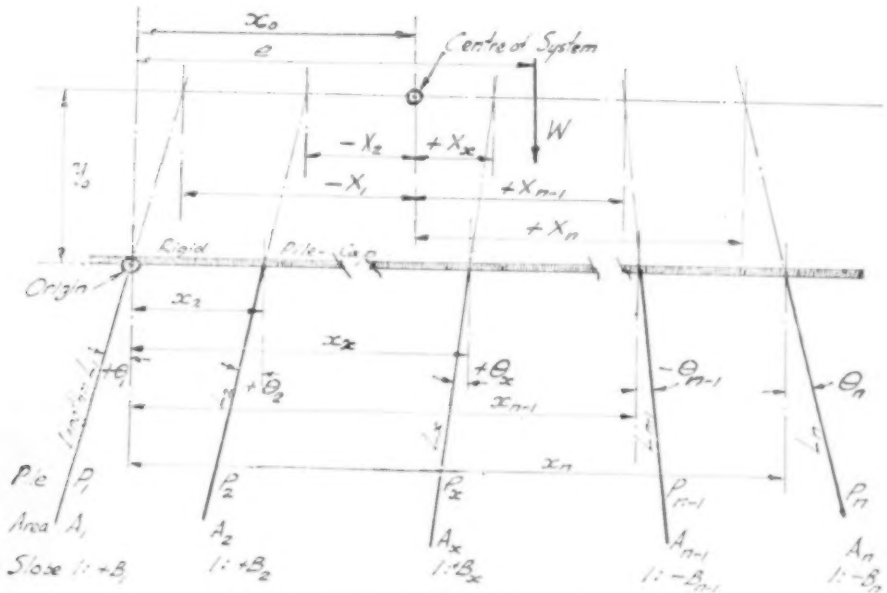

Fig. 2.—Group including Inclined Piles.

which can be written

$$P_x = k_P(k_W W + k_H H + k_M M) \quad . \quad . \quad . \quad . \quad (5a)$$

The moment M is $W(e - x_0) + H(f - y_0)$. The distance X_x is the co-ordinate of the point of intersection of the horizontal through O and the projection of the axis of pile P_x, and is given by $x_x - x_0 + y_0 \tan \theta_x$, which is positive if the intersection is to the right of O. The equivalent moment of inertia I of the group is $\Sigma V_x X^2_x$.

For practical application the foregoing formulæ and supplementary expressions can be simplified and the loads on the piles can be calculated conveniently by tabulation. Consider the constant Q_x. Since all piles in one group are likely to be of the same material, the elastic modulus E can be omitted and $Q_x = \frac{A_x}{L_x}$. If all the piles are of the same cross-section, A_x can be omitted and $Q_x = \frac{1}{L_x}$. If all the piles are of the same length and cross section, Q_x can be considered to be unity.

It is often more convenient to express the inclination θ_x of a pile as a slope of 1 unit horizontally to B units vertically instead of an angle. For a pile inclined towards the right, $\sin \theta_x = \frac{1}{\sqrt{1 + B_x^2}}$, $\cos \theta_x = \frac{B_x}{\sqrt{1 + B_x^2}}$, and $\tan \theta_x = \frac{1}{B_x}$. For a pile inclined towards the left, $\sin \theta_x = -\frac{1}{\sqrt{1 + B_x^2}}$, $\cos \theta_x = \frac{B_x}{\sqrt{1 + B_x^2}}$, and $\tan \theta_x = -\frac{1}{B_x}$. For a vertical pile, $\sin \theta_x = 0$, $\cos \theta_x = 1$, and $\tan \theta_x = 0$. With these modifications, the six summations in the foregoing can be replaced by the following expressions for Σ_1 to Σ_6:

Summation	Piles inclined to the right	Vertical piles	Piles inclined to the left
$\Sigma_1 = \Sigma V_x = \Sigma Q_x \cos^2 \theta_x$	$+\Sigma_R \frac{A_x}{L_x}\left(\frac{B_x^2}{1 + B_x^2}\right)$	$+\Sigma_V \frac{A_x}{L_x}$	$+\Sigma_L \frac{A_x}{L_x}\left(\frac{B_x^2}{1 + B_x^2}\right)$
$\Sigma_2 = \Sigma V_x \tan \theta_x$ $= \Sigma Q_x \cos \theta_x \sin \theta_x$	$+\Sigma_R \frac{A_x}{L_x}\left(\frac{B_x}{1 + B_x^2}\right)$	Nil	$-\Sigma_L \frac{A_x}{L_x}\left(\frac{B_x}{1 + B_x^2}\right)$
$\Sigma_3 = \Sigma V_x x_x = \Sigma x_x Q_x \cos^2 \theta_x$	$+\Sigma_R \frac{A_x}{L_x}\left(\frac{B_x^2 x_x}{1 + B_x^2}\right)$	$+\Sigma_V \frac{A_x}{L_x} x_x$	$+\Sigma_L \frac{A_x}{L_x}\left(\frac{B_x^2 x_x}{1 + B_x^2}\right)$
$\Sigma_4 = \Sigma V_x x_x \tan \theta_x$ $= \Sigma x_x Q_x \cos \theta_x \sin \theta_x$	$+\Sigma_R \frac{A_x}{L_x}\left(\frac{B_x x_x}{1 + B_x^2}\right)$	Nil	$-\Sigma_L \frac{A_x}{L_x}\left(\frac{B_x x_x}{1 + B_x^2}\right)$
$\Sigma_5 = \Sigma V_x \tan^2 \theta_x$ $= \Sigma Q_x \sin^2 \theta_x$	$+\Sigma_R \frac{A_x}{L_x}\left(\frac{1}{1 + B_x^2}\right)$	Nil	$+\Sigma_L \frac{A_x}{L_x}\left(\frac{1}{1 + B_x^2}\right)$
$\Sigma_6 = I = \Sigma V_x X_x^2$ $= \Sigma X_x^2 Q_x \cos^2 \theta_x$	$+\Sigma_R \frac{A_x}{L_x}\left(\frac{B_x^2 X_x^2}{1 + B_x^2}\right)$	$+\Sigma_V \frac{A_x}{L_x} X_x^2$	$+\Sigma_L \frac{A_x}{L_x}\left(\frac{B_x^2 X_x^2}{1 + B_x^2}\right)$

The co-ordinates of O (x_0, y_0) can be expressed in terms of Σ_1 to Σ_5, thus

$$x_0 = (\Sigma_3 \Sigma_5 - \Sigma_2 \Sigma_4) g \; ; \; y_0 = (\Sigma_2 \Sigma_3 - \Sigma_1 \Sigma_4) g \quad . \quad . \quad . \quad (4a)$$

in which $g = \frac{1}{\Sigma_1 \Sigma_5 - \Sigma_2^2}$. In the summations Σ_1 to Σ_6, and therefore in the

value of g, to be substituted in (4*a*), every pile in the group must be considered, that is, the factors for the piles, whether vertical or inclined to the right or to the left, must be included in the totals. The co-ordinate X_x can be evaluated from the following.

$$\left.\begin{array}{ll} \text{Piles inclined towards the right:} & X_x = x_x - x_0 + \dfrac{y_0}{B_x} \\ \text{Vertical piles:} & X_x = x_x - x_0 \\ \text{Piles inclined towards the left:} & X_x = x_x - x_0 - \dfrac{y_0}{B_x} \end{array}\right\} \quad . \quad (6)$$

Similar expressions for the coefficients in formula (5*a*) for the axial load on any pile P_x are the following:

Coefficient	Pile inclined to the right	Vertical pile	Pile inclined to the left
$k_P = Q_x \cos\theta_x = \dfrac{A_x}{L_x}\cos\theta_x$	$+\dfrac{A_x}{L_x}\left(\dfrac{B_x}{\sqrt{1+B_x^2}}\right)$	$+\dfrac{A_x}{L_x}$	$+\dfrac{A_x}{L_x}\left(\dfrac{B_x}{\sqrt{1+B_x^2}}\right)$
$k_W = (\Sigma_5 - \tan\theta_x\Sigma_2)g$	$+\left(\Sigma_5 - \dfrac{\Sigma_2}{B_x}\right)g$	$+\Sigma_5 g$	$+\left(\Sigma_5 + \dfrac{\Sigma_2}{B_x}\right)g$
$k_H = (\tan\theta_x\Sigma_1 - \Sigma_2)g$	$+\left(\dfrac{\Sigma_1}{B_x} - \Sigma_2\right)g$	$-\Sigma_2 g$	$-\left(\dfrac{\Sigma_1}{B_x} + \Sigma_2\right)g$
$k_M = \dfrac{X_x}{I} = \dfrac{X_x}{\Sigma_6}$	$+\dfrac{X_x}{\Sigma_6}$	$+\dfrac{X_x}{\Sigma_6}$	$+\dfrac{X_x}{\Sigma_6}$

Some of the terms can be further simplified if the group of piles is symmetrical; for example, $\Sigma_2 = 0$, $\Sigma_4 = 0$, $x_0 = 0{\cdot}5x_n$ (see *Fig.* 2), $y_0 = 0$, and $g = \dfrac{1}{\Sigma_1\Sigma_5}$; Σ_3 is not required.

The method of applying these formulæ and expressions to calculate the load on a pile is to evaluate the summations Σ_1 to Σ_6, and, by substitution, to calculate g and the ordinates x_0 and y_0 of the centre O (formulæ 4*a*). The value of X_x for each pile can then be determined from formulæ (6). Sufficient data are now known to evaluate the coefficients k_P, k_W, k_H, and k_M for each pile and, by substitution in formula (5*a*), the axial load on each pile can be determined.

Appendix I.—Groups containing Vertical Piles only.

Referring to *Fig.* 1, the distance X of the centroid of the group from the left-hand pile is $\dfrac{\Sigma aN}{\Sigma N}$; ΣN is the total number of piles in the group, and for a symmetrical group $X = 0{\cdot}5a_n$. The equivalent moment of inertia I of the group about the centroid is ΣNx^2. The axial load on any pile due to W alone is $\dfrac{W}{\Sigma N}$, to which must be added the weight W_x of the pile. The horizontal force H produces a total anticlockwise moment Hh on the group. The eccentricity of the total load W produces a clockwise moment We if W is to the right of the centroid of the group, that is if e is positive; the moment is anticlockwise, that is $-We$, if W is to the left of the centroid. The net moment M on the group is therefore $We - Hh$ (clockwise if positive, anticlockwise if negative), and the consequent axial load on each pile in a row containing N_x piles each at a distance $+x_x$ from the centroid of the group is $\dfrac{Mx_x}{N_xI}$. The total axial load

P_x on any pile is therefore given by

$$P_x = \frac{W}{\Sigma N} + \frac{x_x(We - Hh)}{N_x I} + W_x$$

If the horizontal force is resisted equally by each pile, the shearing force S_x on each pile is $\frac{H}{\Sigma N}$ and the bending moment on each pile, assuming fixity at the head and at datum level, is $0{\cdot}5hS_x$, that is $\frac{hH}{2\Sigma N}$. Care must be taken with the algebraical sign of each quantity and each term.

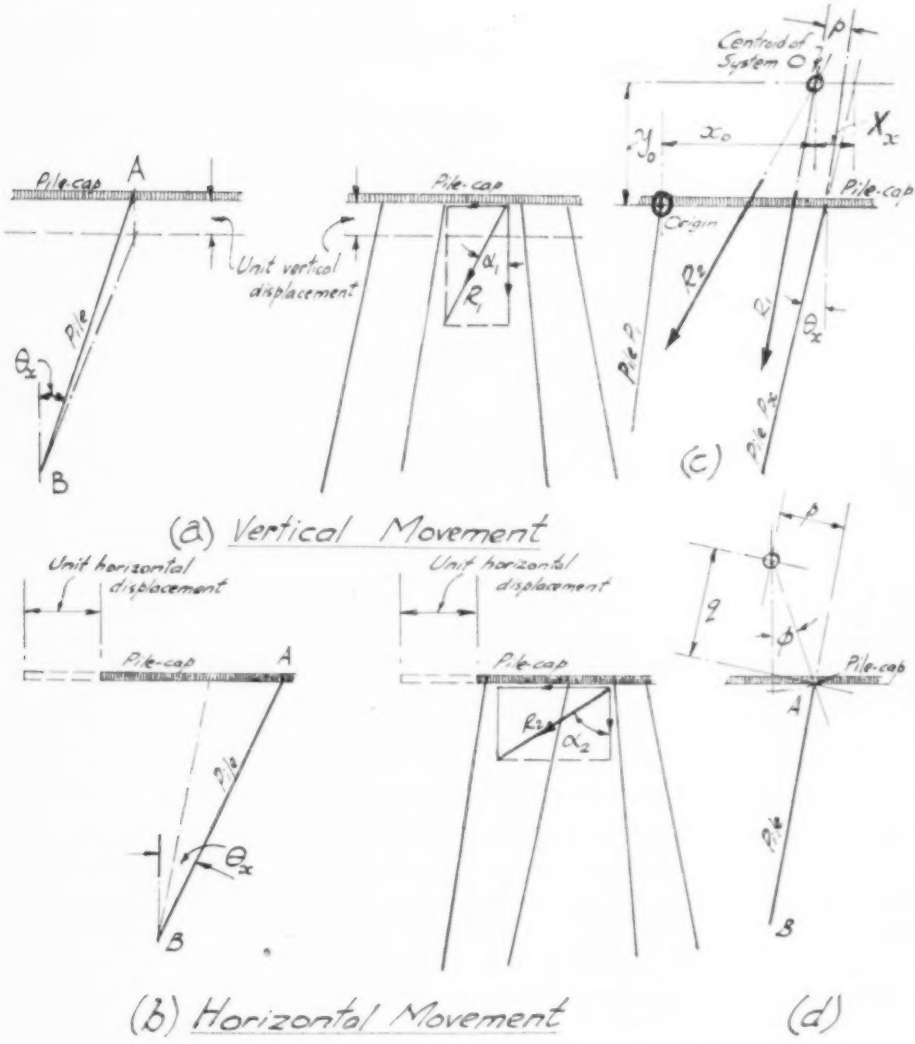

Fig. 3.—Analysis of Group including Inclined Piles.

Appendix II.—Groups containing Inclined Piles.

The ordinary theory for evaluating the loads on piles in groups containing inclined piles, with or without vertical piles, is given in various text books, for example, "Reinforced Concrete" by Professor A. L. L. Baker. The following is based on the analysis given in "Cours de Stabilité des Constructions," Volume II, by Professor G. Magnel.

If a non-deformable pile-cap is displaced unit distance vertically downwards (*Fig.* 3*a*), the axial deformation of pile AB is $\cos\theta_x$ and the reaction of the pile is $Q_x \cos\theta_x$ if $Q_x = \frac{EA_x}{L_x}$. The vertical component of this reaction is $Q_x \cos^2\theta_x (= V_x)$ and the horizontal component is $Q_x \cos\theta_x \sin\theta_x$, that is $V_x \tan\theta_x$. The value of θ_x is considered to be positive if the pile is inclined towards the right, and the horizontal component is positive if it acts towards the right. The resultant of the vertical components for all piles in the group is ΣV_x and that of the horizontal components is $\Sigma V_x \tan\theta_x$. These resultants can be replaced by the reaction $-R_1$, which must be equal to the contrary force R_1 which produces unit vertical movement. The angle α_1 between the line of action of R_1 and the vertical is such that $\tan\alpha_1 = \frac{\Sigma V_x \tan\theta_x}{\Sigma V_x}$, say, α. The magnitude of R_1 is $\frac{\Sigma V_x}{\cos\alpha}$.

Similarly unit horizontal movement of the pile-cap towards the left (*Fig.* 3*b*) produces an axial displacement of the pile equal to $\sin\theta_x$ and the reaction of the pile is $Q_x \sin\theta_x$, the vertical and horizontal components of this reaction being $V_x \tan\theta_x$ and $V_x \tan^2\theta_x$ respectively. For all the piles in the group the components of the force R_2, which produces these reactions, are $\Sigma V_x \tan\theta_x$ vertically and $\Sigma V_x \tan^2\theta_x$ horizontally. The inclination of R_2 is such that $\tan\alpha_2 = \frac{\Sigma V_x \tan^2\theta_x}{\Sigma V_x \tan\theta_x}$, say, β. The magnitude of R_2 is $\frac{\Sigma V_x \tan\theta_x}{\cos\beta}$.

Consider, as in *Fig.* 3*c*, that the lines of action of the two forces R_1 and R_2 intersect at O. If a non-deformable pile-cap rotates clockwise about O through an angle ϕ, the axial and lateral displacements of the pile-head are $p\phi$ and $q\phi$ respectively (*Fig.* 3*d*). The axial displacement produces a reaction $Q_x p\phi$, the vertical component of which is $Q_x p\phi \cos\theta_x$, that is $\frac{p\phi V_x}{\cos\theta_x}$ or $\phi V_x X_x$ if X_x is the distance from O to the intersection of the axis of the pile (produced) and the horizontal through O (*Fig.* 2); X_x is positive if the intersection is to the right of O. Similarly the horizontal component is $Q_x p\phi \sin\theta_x$, that is $\frac{p\phi V_x \sin\theta_x}{\cos^2\theta_x}$ or $\phi V_x X_x \tan\theta_x$. Since O is on the lines of action of R_1 and R_2, $\Sigma V_x X_x = 0$ and $\Sigma V_x X_x \tan\theta_x = 0$. Therefore $\phi\Sigma V_x X_x = 0$ and $\phi\Sigma V_x X_x \tan\theta_x = 0$, and consequently the effect capable of producing rotation ϕ is a couple M of magnitude $\Sigma Q_x p^2\phi$, that is $\phi\Sigma Q_x X_x^2 \cos^2\theta_x$ or $\phi\Sigma V_x X_x^2$. If $\Sigma V_x X_x^2 = I$ (the effective moment of inertia of the group), $M = \phi I$ which is positive if clockwise.

Considering the head of the left-hand pile as the origin (*Fig.* 2), the distance to the point of application of R_1 is $\frac{\Sigma V_x x_x}{\Sigma V_x}$; therefore $y = \left(x - \frac{\Sigma V_x x_x}{\Sigma V_x}\right)\frac{1}{\alpha}$; similarly for R_2, $y = \left(x - \frac{\Sigma V_x x_x \tan\theta_x}{\Sigma V_x \tan\theta_x}\right)\frac{1}{\beta}$. The co-ordinates of O are therefore

$$\left.\begin{aligned} x_0 &= \frac{\frac{\beta\Sigma V_x x_x}{\alpha\Sigma V_x} - \frac{\Sigma V_x x_x \tan\theta_x}{\Sigma V_x \tan\theta_x}}{\frac{\beta}{\alpha} - 1} = \frac{\beta\frac{\Sigma V_x x_x}{\Sigma V_x} - \alpha\frac{\Sigma V_x x_x \tan\theta_x}{\Sigma V_x \tan\theta_x}}{\beta - \alpha} \\ y_0 &= \frac{\frac{\Sigma V_x x_x}{\Sigma V_x} - \frac{\Sigma V_x x_x \tan\theta_x}{\Sigma V_x \tan\theta_x}}{\beta - \alpha} \end{aligned}\right\} \quad . \quad . \quad (4)$$

Now consider the effect of external forces acting in the plane of the piles. These forces can be represented by a force F_1 in the line of R_1, a force F_2 in the line of R_2 and a couple M. Since R_1 and R_2 produce reactions in any pile of V_x and $V_x \tan \theta_x$ respectively, F_1 and F_2 produce reactions of $\frac{F_1}{R_1} V_x$ and $\frac{F_2}{R_2} V_x \tan \theta_x$ respectively. The rotation ϕ due to M is $\frac{M}{I}$ and produces a reaction $Q_x X_x \cos \theta_x \phi$, that is $V_x X_x \frac{M}{I}$. Therefore the total load P_x carried by any pile is $\frac{V_x}{\cos \theta_x}\left(\frac{F_1}{R_1} + \frac{F_2}{R_2} \tan \theta_x + \frac{MX_x}{I}\right)$. Substituting the values of R_1 and R_2 gives

$$P_x = \frac{V_x}{\cos \theta_x}\left(\frac{F_1 \cos \alpha_1}{\Sigma V_x} + \frac{F_2 \cos \alpha_2 \tan \theta_x}{\Sigma V_x \tan \theta_x} + \frac{MX_x}{I}\right).$$

For any pile, the distance X_x is $x_x - x_0 + y \tan \theta_x$.

In general, the external forces are defined by their vertical component W and horizontal component H, and therefore $W = F_1 \cos \alpha_1 + F_2 \cos \alpha_2$ and $H = F_2 \sin \alpha_1 + F_2 \sin \alpha_2$; W is positive if downwards and H is positive if towards the left. Substitution in the formula for P_x gives

$$P_x = \frac{V_x}{\cos \theta_x}\left[\frac{W(\beta - \tan \theta_x)}{(\beta - \alpha)\Sigma V_x} + \frac{H(\tan \theta_x - \alpha)}{(\beta - \alpha)\Sigma V_x \tan \theta_x} + \frac{MX_x}{I}\right] \quad . \quad . \quad (5)$$

to which must be added the weight of the pile.

Concrete Ship used as a Caisson.

In the construction of a new harbour at St. Galini, Crete, a concrete ship built in Great Britain in the year 1919 was used as a caisson to form a breakwater behind which floating equipment could be used to build the remainder of the mole. The work, described in "The Dock and Harbour Authority" for March, 1953, included building within the ship transverse and longitudinal stiffening walls dividing it into eighteen compartments and raising the height of the deck by about 9 ft. This was done in the harbour of Piræus, on the coast of Greece, from whence the ship was towed to St. Galini.

As the ship was about 167 ft. long it was necessary to avoid unequal settlement of the sea-bed where the ship was to be submerged and to form a level surface on which the ship would rest. Consequently a layer of broken rock, about 3 ft. thick, was placed on the sea-bed. The ship was held in position above this base by four anchors and submerged by opening valves in the eighteen compartments. The sinking was completed in three hours, and in the following twelve days the ship was filled with about 4000 cu. yd. of broken stone. A plan showing the ship in position in the mole is shown in *Fig.* 1.

The remainder of the mole is formed of 2600 cu. yd. of large precast concrete blocks placed in position by floating cranes. The breakwater on the seaward side is of broken rock of which about 13,000 cu. yd. were required.

The work was carried out for the Greek Ministry of Public Works by the Archimidis Contracting Co., who sublet the contract for the reinforced concrete work in the ship to the N. Gavalas Contracting Co.

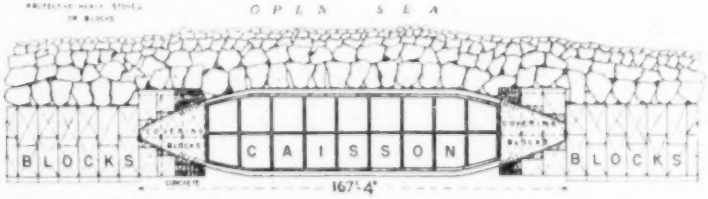

Fig. 1.—Plan of Mole with Ship Forming Caisson in Position.

Precast Construction in Denmark—1.

THE Institution of Civil Engineers of Denmark recently invited some of its members to contribute papers on the design, manufacture, and erection of precast concrete structural members. These papers are published in full in the Danish language in " Beton og Jernbeton ", No. 3, 1953, and also in " Betonteknik ", No. 1, 1953. The following abstracts are given by permission.

A LARGE INDUSTRIAL BUILDING.

By R. A. Larsen.

Large industrial buildings for the Nordisk Kabel og Traadfabrikker at Glostrup, Denmark, have a total roofed area of about 430,000 sq. ft.; the owner required a standard width of 55 ft. for the bays but the height varies in the different buildings. In most of the buildings girders for overhead cranes are provided; for this reason services and ducts are along the lines of columns. One-fourth of the skylights have openable frames. The columns were cast in situ at 29 ft. 6 in. centres. The rest of the building consists essentially of ten types of precast units, with a few special members at corners and near expansion joints.

ROOF.—The roof (*Fig.* 1) comprises four types of members. The trusses are simply-supported Vierendeel girders prestressed prior to erection. They are 4 ft. 6 in. high, 1 ft. 3 in. wide, and weigh about 10 tons. The beams at each side of the skylight are reinforced to carry their own weight; after erection they were prestressed in the length of the building (200 ft. to 260 ft.) to provide continuity. The same principle was used

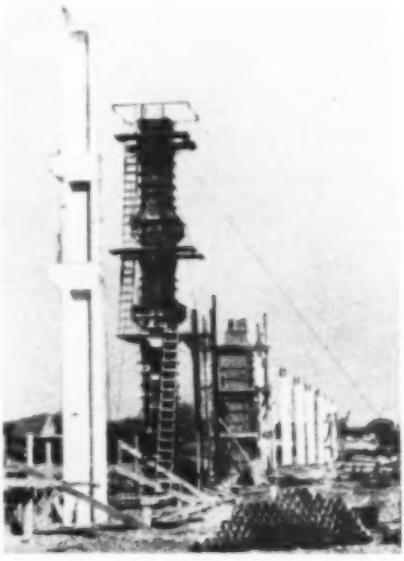

Fig. 2.—Casting the Columns.

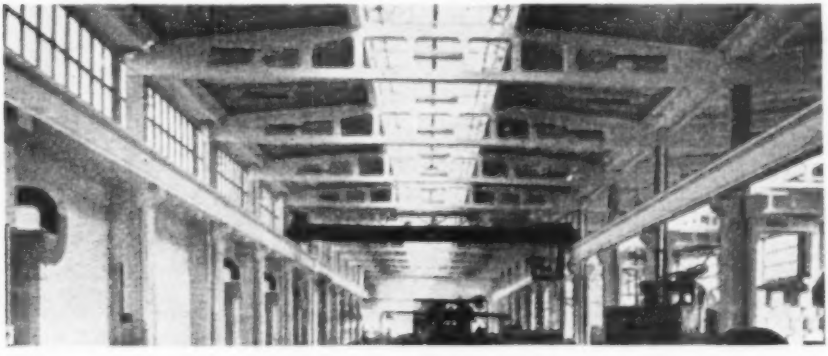

Fig. 1.—Roof Supported by Prestressed Precast Vierendeel Girders.

Fig. 3.—Walls of Precast Slabs.

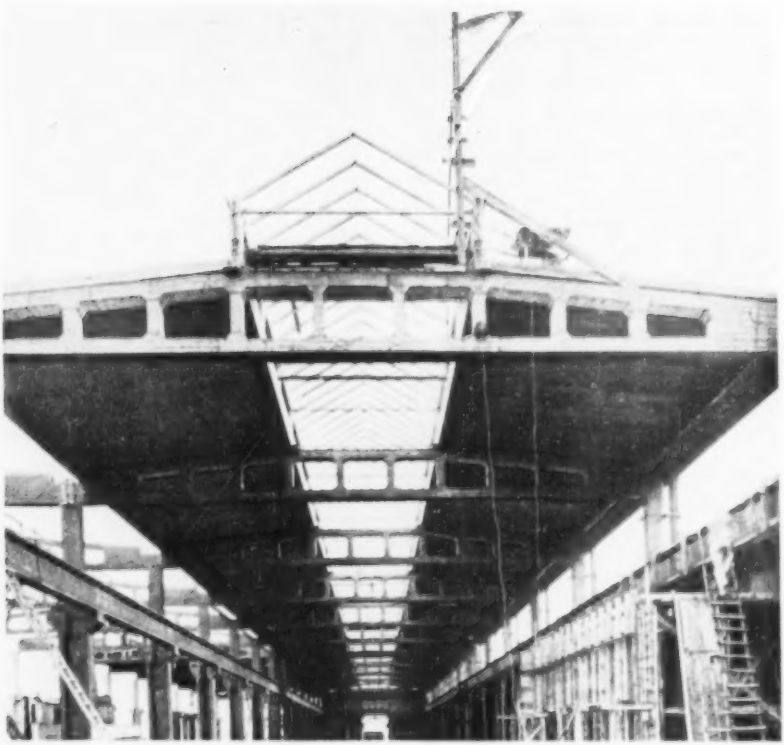

Fig. 4.—The Building during Erection.

for the crane-beams and valley-beams. The slabs are supported on the skylight-beams and valley-beams. The slabs are $1\frac{5}{8}$ in. thick with ribs dividing them into panels 4 ft. 6 in. by 5 ft. 9 in.; the stiffening ribs are $7\frac{1}{8}$ in. deep.

WALLS.—The external walls (*Fig.* 3) are of cavity construction. There are four slabs about 18 ft. by 7 ft. in each bay and a pilaster at each column. The inner leaf is of clinker concrete $4\frac{3}{8}$ in. thick. The outer leaf of dense concrete has a patterned face $1\frac{1}{4}$ in. thick, and a backing of the same thickness. On top of the slabs is a lintel, which also serves as wind-bracing. The concrete window-frames measure 5 ft. 7 in. by 10 ft. 6 in. The lintels are 30 ft. long. The concrete for the pilasters and lintels is coloured and was treated with acid to expose the aggregate.

Fig. 5.—Erecting Wall Slabs.

Fig. 6.—Erection of Beam for Skylight.

COLUMNS.—The columns are I-shape in cross section, and in the recesses are drain pipes and electric mains. The columns were cast in situ in 20-ft. lifts, shuttering being bolted to the footings (*Fig.* 2). The concrete was placed by the cranes used for erecting the precast members.

MAKING THE MEMBERS.—The work was large enough to justify precasting on the site. The factory was in the middle of the site and could produce each week enough members for buildings covering 4000 to 5000 sq. ft. Timber moulds were used. Each mould was used about a hundred times, and the repairs required were negligible. The time required to make a casting varied from two days (at a temperature of 65 deg. F.) for roof slabs to six days for prestressed main beams. The slabs were cast horizontally and removed from the moulds by a vacuum lifter which also erected them into a vertical position for curing. During subsequent transport the slabs remained vertical. The slabs were removed from the moulds by tongs after removing the sides of the moulds.

ERECTION.—The main girders were erected by vertical booms, because a 10-tons crane was not available. Travelling cranes were used for the erection of the other members. In designing the members the lifting capacity of the cranes was taken into consideration, while making the members as large as possible in order to reduce the number. With the exception of the main girders the weight of all the members is between $2\frac{1}{2}$ and 4 tons. The wall slabs were erected independently of the roof. *Figs.* 4 to 6 show the methods. The triangular frames for the skylight are prestressed. The roof is insulated with woodwool slabs, placed after the erection, with a surface rendering and two layers of roofing felt.

Wood-wool slabs were placed between the valley girders and served as bottom shutters for the valleys. The windows were inserted when the walls were completed. All joints are grouted. The surfaces are as left from the shuttering and moulds, and are cement washed. The erection was done at the same rate as the production of the members, namely, 4000 to 5000 sq. ft. per week.

HOUSES AND FACTORIES.

By N. J. Manniche.

"HJORTEKAER" HOUSES.—About 200 one-story houses of this type were built for the Workmen's Co-operative Housing Association during the years 1947 and 1948 when it was not possible to make use of precast units to the same extent as in subsequent schemes. The external walls have a precast reinforced concrete frame. Windows and doors (*Fig.* 7) are fixed in precast frames, for which white cement was used. These frames include the exterior sill and architrave. The window frames are carried down to the foundation so that the wall is divided into panels with expansion joints to reduce the effects of changes of temperature. The frames support external and internal woodwool slabs 1⅜ in. thick, the external slab having a rendering of three coats of mortar reinforced with expanded metal at the joints and at the corners of the house. The roof slab is reinforced with wire mesh and insulated with 3-in. woodwool slabs on the underside. Expansion joints are provided above partition walls. The roof is supported by the walls but is separated from them by two layers of bituminous felt to allow for expansion and contraction.

"HERLEV" HOUSES.—In the years 1948–49 about 300 single-story houses were built for the Copenhagen Building Society, fifty having reinforced concrete frames and 250 load-bearing walls. In

Fig. 7.—Window and Door Panels of Hjortekaer House.

the framed houses the walls are precast slabs 2¼ in. thick with stiffening ribs fixed to the frame by vertical dovetail joints reinforced with U-shaped stirrups. The internal lining is 3-in. woodwool slabs, which are plastered.

The houses (*Fig.* 8) with load-bearing walls comprise precast panels 3 ft. 8 in. wide by 8 ft. 6 in. high by 3 in. thick, with a 2-in. insulating layer attached; there are ten cylindrical holes in each slab as shown in the cross section in

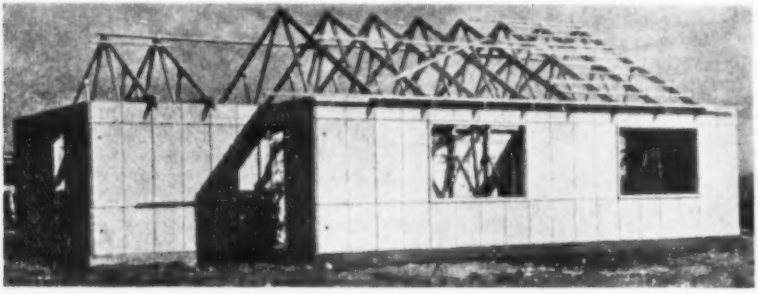

Fig. 8.—Erection of Herlev Houses.

Fig. 9. The slabs were made on a production line; partition walls were cast on the floor and erected by the tilt-up method. The roofs are of timber and corrugated asbestos-cement sheets.

BARRACKS.—In the construction of single-story barracks with a roofed area of 160,000 sq. ft. the structures were precast with surfaces that required no further treatment or which were suitable for painting. Slabs for roofs and external walls have an insulation of rock-wool or glass-wool; a section through a wall slab is shown in *Fig.* 10. The roof slabs have an upper layer of dense reinforced concrete, a core of rock-wool, and a lower layer of lightweight concrete. All partition walls are of lightweight concrete precast to the full height. The slabs were made on the site under cover. Erection was done by cranes. Precasting started in July, 1952, and nearly all buildings were completed by the end of the year.

BOILER SHOP.—In 1951 a boiler shop 410 ft. long (*Figs.* 11 and 12) was built at the shipyard of Messrs. Burmeister and Wain. Precast members were used except for the main columns, which were cast in situ. The arches supporting the roof are of timber. The crane girders

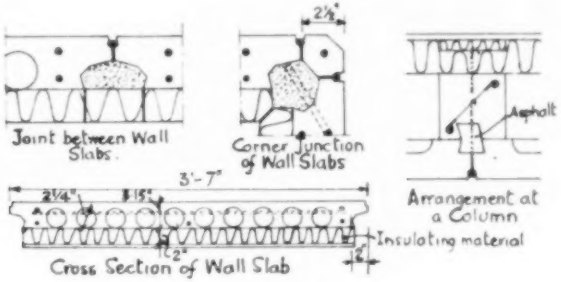

Fig. 9.—Sections of Wall Slabs for Houses.

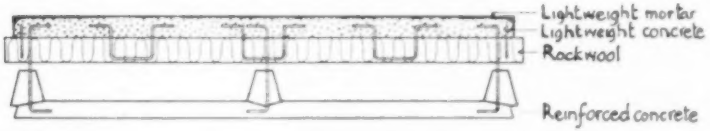

Fig. 10.—Wall-slabs for Barracks.

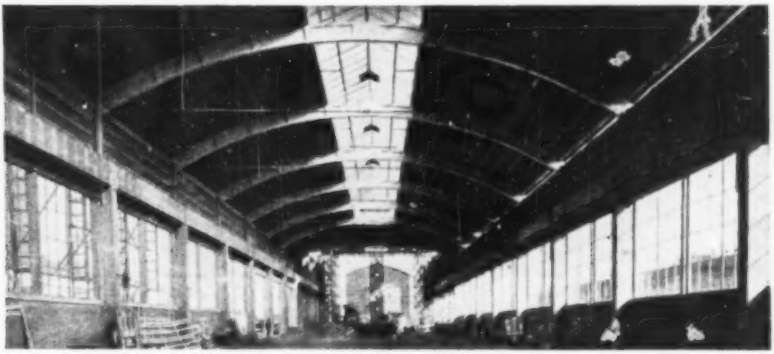

Fig. 11.—Interior of a Boiler Shop.

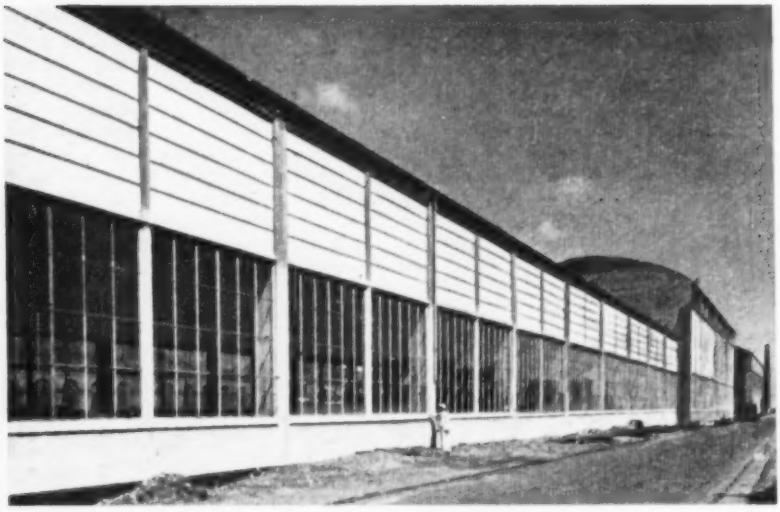

Fig. 12.—Wall of Boiler Shop.

Fig. 13.—A Complete Prefabricated Factory.

were cast on the floor; they are channel-shaped beams with 2¼-in. webs and flanges. After erection on the columns, additional reinforcement was placed and the channels filled with in-situ concrete. The crane girders have a cross section of 1 ft. 8 in. by 4 ft. 2 in. and are 25 ft. long. The weight of a complete girder is about 12 tons. The weight of each precast channel beam was 3 tons. The walls and gables have secondary ribs and panels about 12 ft. long. All the members were erected by a tower-crane travelling on rails at the centre of the building and covering the entire site. Sixteen scaffold towers, moved by the tower-crane, were used for the construction of the 34 main columns, two scaffolds being used for each column. Each scaffold tower weighed about 1¼ tons and could be moved by the crane in about ten minutes. The amount of timber used for the scaffold and shuttering was much less than that generally used for construction of this kind.

Fig. 14.—Joints between Roof Beams and Columns

Figs. 13 and 14 give an interior view and a detail of another precast factory recently erected in Denmark.

(To be continued.)

Test of Concrete-and-Glass Panels.

At the Building Research Station two panels comprising reinforced concrete beams and glass lenses have been tested. The panels measured 11 ft. 3 in. by 4 ft. 2 in., and were designed and made by different firms. Each panel consisted of seven longitudinal ribs 3 in. deep at 8-in. centres reinforced with one ½-in. diameter bar in the bottom, and fourteen similar transverse ribs at 8-in. centres each reinforced with one $\frac{5}{16}$-in. bar. In one panel glass lenses 6½ in. by 6½ in. by ⅞ in. had been placed in position before the concrete was placed. In the other panel, plaster was used instead of glass and the plaster broken out before the test. Both panels were simply supported on walls 10 ft. 3 in. apart and loads applied until failure occurred. The deflection of the panel with lenses was about one-half of that of the other under similar loading, and the lenses so strengthened the compressive zone of the panel that the load could be increased until yield occurred in the reinforcement. Failure was thus slow and progressive, whereas the panel without lenses failed suddenly due to compression.

Concrete Pumped to a Height of 176 ft.

During the construction of the Mayo Memorial Medical Centre at the University of Minnesota in Minneapolis, U.S.A., concrete was pumped to a height of 176 ft. 4 in. for the construction of the roof of a three-story structure above the seventeen stories of the main building.

The building is a steel-framed structure with concrete floors. Two machines were used to pump the concrete through 6-in. pipes which were supported at intervals of 20 ft. by frames tied to the steel frame. The greatest length of pipe from the pumps to the point of placing was 550 ft. For the lower floors the two machines supplied up to 34 cu. yd. of concrete per hour, but for the upper floors this was reduced to an average of 15 cu. yd. per hour. Nevertheless, from the seventh to the seventeenth floor it was possible to complete one floor, containing 350 cu. yd. of concrete, each week with eight men only. Altogether 23,000 cu. yd. of concrete were required for the building.

The building was designed by C. H. Johnson Architects-Engineers, and the contractors were the Steenberg-Madsen Construction Co. The foregoing is abstracted from "Engineering News-Record" for August 27, 1953.

A Prestressed Precast Helicoidal Staircase.

THE helicoidal staircase shown in *Fig.* 1 comprises nineteen precast steps, each of the same shape, connected together by three cables anchored in in-situ members at the top and bottom. The height of the staircase is 13 ft. 1½ in. and its diameter on a horizontal projection is 6 ft. 10½ in. The treads are 2 ft. 3½ in. long and have a width varying from about 4 in. to 13 in.; the rise is about 7⅞ in. The shape of the steps is shown in *Figs.* 2 and 3, and a developed section is given in *Fig.* 4 (these illustrations are on page 389).

In each step are formed three holes of 1⅜ in. diameter, except for 3 ft. 3 in. at the top and bottom of the staircase where they are 2 in. diameter, through which are passed the cables. Each cable comprises twelve 5-mm. wires which are stressed to about 50,000 lb. per square inch giving a total force for the three cables of about 23 tons. The cables are anchored at the first and last step by steel cylinders and wedges of a patented Italian design and the ends of the wires projecting from the anchors are embedded in the top and bottom in-situ members.

The steps were cast in a single mould and were threaded on to the cables, the lower ends of which had been cast in the springing member. The cables were tensioned from the top step by a jack of Italian manufacture. After the cavities for the cables had been grouted the landing member was cast.

The staircase was tested by applying a load of 220 lb. on each step, and under this load the vertical deflection at the mid-height of the staircase was 0·006 in.

Fig. 1.

The staircase was designed by Dr. Giuseppe Rinaldi and is described in the Spanish journal "Informes de la Construccion" for December, 1952.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Field Maintenance of Builders' Plant, by J. Stafford (George Wimpey & Co., Ltd.). Ministry of Works Building, Ashley Street, Birmingham 5. December 15. 7.15 p.m.

Foundation Problems, by L. R. Creasy (Superintending Structural Engineer, Ministry of Works). Technical College, Stoke Park, Guildford. December 16. 7 p.m.

Scottish Building Practice and Techniques, by D. K. Baron (Building Research Station). Oughton's Restaurant, Church Place, Dumfries. December 17. 7.15 p.m.

Productivity in House Building, by A. E. Chittenden (Ministry of Works). Technical College, Hampstead Road, Watford. December 17. 7 p.m.

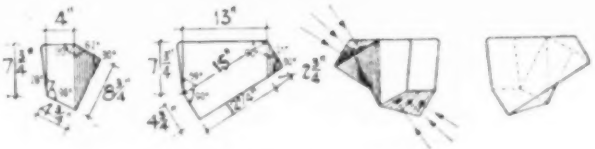

Fig. 2.—Details of a Step.

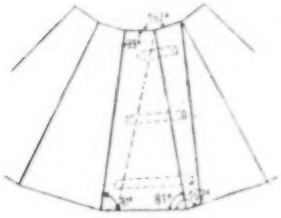

Fig. 3.—Plan of Steps.

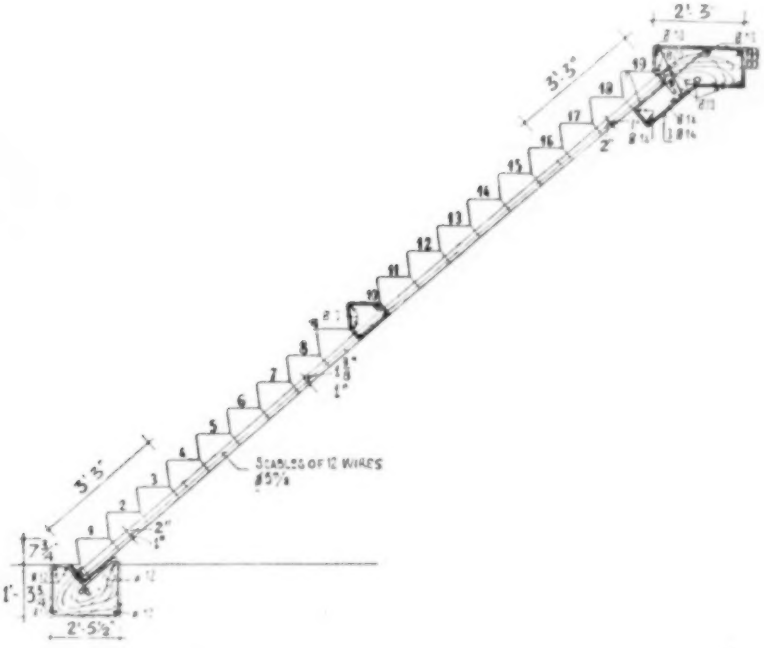

Fig. 4.—Developed Section through the Staircase.

Details of Prestressed Precast Helicoidal Staircase.

(*See page* 388.)

Book Reviews.

"Theory and Practice of Reinforced Concrete." By C. W. Dunham. (London: McGraw-Hill Book Co. 1953. Price 56s.)

THE third edition of this book is in two volumes, of which this (the first) deals with the principles of the design and analysis of reinforced concrete members. Statically-indeterminate frames are not considered in this volume. The treatment is in accordance with modern U.S.A. practice and conforms with the building code, published in 1951, of the American Concrete Institute. Methods of design based upon loads at failure are explained, and there is a chapter on prestressed concrete.

The theories are demonstrated by applying them to the design of simply-supported beams and slabs, foundations, and walls. A method of calculating the pressure on a retaining wall due to a concentrated surcharge is also given. The design of slabs reinforced in two directions and subjected to a uniformly-distributed load is treated briefly, as also are flat-slabs.

Chapters deal with the properties and manufacture of concrete, shuttering (including data on the pressure due to wet concrete), and miscellaneous details including surface finishes, waterproofing, and construction, expansion, and contraction joints.

This volume is suitable for students rather than practising engineers, and there are many completely worked-out numerical examples and problems to which the answers are given.

"Calculation, Design and Testing of Reinforced Concrete." By K. L. Rao. Second Edition. (London: Sir Isaac Pitman & Sons, Ltd. Price 35s.)

THIS edition of this book differs from the first only in that short notes have been added on the ultimate strength of singly-reinforced beams, rigid frames, reinforcement around large openings, and shell roofs. The code of practice published in 1934 by the D.S.I.R. is still referred to as the "new" code and references to the by-laws of the London County Council are to the by-laws which were superseded in 1952.

In presenting in one volume the theory of reinforced concrete and the testing of materials and simple structural members, the book fulfils its purpose admirably. The part dealing with design would, however, be improved for the use of students by more and clearer drawings showing the arrangement of reinforcement in simpler structures in place of details, which necessarily must be inadequate, of structures such as shell roofs.

"Die Bodenphysikalischen Grundlagen Stützmauern." **"Handbuch für Stahlbetonbau."** Vol. IV, Part 1. Fifth edition. By Otto Mund. (Berlin: Wilhelm Ernst & Sohn. 1953. Price 27 DM.)

THIS book, which forms part of the standard handbook on reinforced concrete in the German language, deals with the principles of soil mechanics and the design of retaining walls.

The author introduces the theory of stability of walls subjected to earth pressure, giving first Coulomb's wedge theory and then a detailed treatment of graphical methods derived therefrom, including the solution by Rebhann. This method is developed to determine in a simple manner, by application of planes of rupture in each case, the distribution of pressure on walls subject to any combination of surcharge. Rankine-Winkler's theory of earth pressure, based upon the conception of a mass of earth whose upper surface is indefinitely extended, is examined and is stated to give accurate results only where the slope of the soil coincides with its natural angle of repose. It is stated that this theory cannot therefore be generally applied to gravity walls, but can always be relied on, as has been demonstrated by Mörsch, to give correct results for walls of the cantilever type with or without counterforts that are usual in reinforced concrete. Numerous clear diagrams are a feature of the book.

Examples of construction have been taken from the previous edition and many interesting recent works have been added. These include bridge abutments and deep foundations, as well as a wide range of plain and reinforced concrete retaining walls. The examples are illustrated by valuable details of construction. Retaining walls subject to hydrostatic pressure by groundwater or a water-table are also considered.

"British Standards Institution Year Book." 1953. (London: British Standards Institution. Price 12s. 6d.)

THIS year-book contains a list of the British Standards current at March 31, 1953, and gives a brief description of each Standard. It also contains a list of British Standard Codes of Practice for buildings and the names of the members of the main councils and committees.

Analysis of a Parabolic Arch Supported by Columns Fixed at their Bases.—II.*

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

Example of Design.

THE weights of the arch rib, purlins, glazing, and sheeting will be considered as uniformly distributed over the horizontal projection of the centre-line of the arch rib. A more accurate investigation can be made by influence lines plotted from the equations previously derived, multiplying the ordinates of the influence lines by the actual loads or by the loads acting at the centres of gravity of the segments of the rib.

The dimensions of the frame are assumed to be: Spacing, centre to centre, of frames, $s = 15$ ft.; column height $= h =$ span of arch rib $= L = 20$ ft.; rise of arch $= r = \frac{L}{2} = 10$ ft. The rib at the crown is 9·5 in. deep by 10 in. wide, $I_0 = \frac{860}{1{\cdot}2}$. The rib at the springings is 12·5 in. deep by 10 in. wide, $I_s = \frac{1953}{1{\cdot}2}$. The column is 12 in. deep by 10 in. wide, $I_c = \frac{1720}{1{\cdot}2}$. Therefore $I_c = 2I_0$, and $I_s = 2{\cdot}27I_0$, which is as near as is practicable to $2{\cdot}24I_0$.

The concrete is ordinary-grade and vibrated, the mixture being 1 : 2 : 4. The maximum allowable stresses in bending are 1100 lb. per square inch compression in concrete, and 18,000 lb. per square inch tension in steel. In accordance with British Standard Code of Practice No. 114 (1948) these stresses may be increased by one-third for the effects due to wind.

		lb. per sq. ft.
DEAD LOAD.—Concrete purlins		4
Average weight of sheets and glazing		5
Average weight of arch rib		11
	Total dead load w_1,	20
Columns		8
Wall sheeting		3½
Steel side rails		2½
	Total =	14

SUPERIMPOSED LOAD.—In accordance with British Standard Code of Practice No. 3, Chapter 5 (1952), the superimposed load varies with the slope of the roof from 15 lb. per square foot for slopes up to 30 deg. to zero at a slope of 75 deg. For this case an average load is assumed of 10 lb. per square foot of the horizontal projection of the roof.

Assuming normal openings in the walls in accordance with C.P. No. 3, Chapter 5, and a maximum velocity of the wind of 63 miles per hour, the horizontal load p due to wind is 10 lb. per square foot. By dividing the curved roof into four segments by chords at angles of approximately 60 deg. and 30 deg. to the horizontal, average pressures on the vertical projection r may be obtained

* Part I was given in this Journal for August, 1953.

of $0{\cdot}4p$ and $-0{\cdot}1p$ on the windward side and $-0{\cdot}45p$ on the leeward side. The horizontal components give average pressures of $0{\cdot}3p \pm 0{\cdot}2p$ and $-0{\cdot}45p \pm 0{\cdot}2p$ on the windward and leeward sides respectively. For walls with normal openings the pressures may be assumed to be $0{\cdot}5p \mp 0{\cdot}2p$ on the windward side and $-0{\cdot}5p \pm 0{\cdot}2p$ on the leeward side. As the vertical suctions due to wind are less than the dead load of the roof they will be ignored.

Three types of load, as shown in *Fig.* 1, are considered, namely, (*a*) Horizontal loads, $ps = 150$ lb. per foot of height; the maximum moments, etc., are obtained from $p_1 = (0{\cdot}5 \pm 0{\cdot}2)p$; (*b*) The horizontal component of the normal wind load acting on the vertical projection of the roof, $ps = 150$ lb. per foot of height, the maximum moments being due to pressures $p_2 = (0{\cdot}3 \pm 0{\cdot}2)p$ and $-0{\cdot}45p \pm 0{\cdot}2p$ on the windward and the leeward sides respectively; (*c*) The vertical load ws due to the dead load $w_1s = 300$ lb. per foot of span and to the imposed load $w_2s = 150$ lb. per foot of span.

$$\Sigma \frac{M_{AB}}{sL^2} = \frac{238}{840}p_1 + \frac{132{,}861}{730{,}240}p_2 - \frac{13w}{326} = 0{\cdot}284p_1 + 0{\cdot}182p_2 - 0{\cdot}0399w.$$

$$\Sigma \frac{M_{BA}}{sL^2} = \frac{78{\cdot}5}{840}p_1 + \frac{77{,}219}{730{,}240}p_2 - \frac{16w}{326} = 0{\cdot}0934p_1 + 0{\cdot}106p_2 - 0{\cdot}0491w.$$

$$\Sigma \frac{M_{CD}}{sL^2} = \frac{26}{840}p_1 + \frac{48{,}291}{730{,}240}p_2 + \frac{16w}{326} = 0{\cdot}031p_1 + 0{\cdot}066p_2 + 0{\cdot}0491w.$$

$$\Sigma \frac{M_{DC}}{sL^2} = \frac{77}{840}p_1 + \frac{106{,}749}{730{,}240}p_2 + \frac{13w}{326} = 0{\cdot}0916p_1 + 0{\cdot}146p_2 + 0{\cdot}0399w.$$

$$\Sigma \frac{H_{1A}}{sL} = \frac{737}{840}p_1 + \frac{5252}{18{,}256}p_2 - \frac{29w}{326} = 0{\cdot}877p_1 + 0{\cdot}288p_2 + 0{\cdot}089w.$$

$$\Sigma \frac{H_{1B}}{sL} = \frac{103}{840}p_1 - \frac{5252}{18{,}256}p_2 + \frac{29w}{326} = 0{\cdot}1226p_1 - 0{\cdot}288p_2 + 0{\cdot}089w.$$

$$\Sigma \frac{H_{2C}}{sL} = \frac{103}{840}p_1 + \frac{3876}{18{,}256}p_2 + \frac{29w}{326} = 0{\cdot}1226p_1 + 0{\cdot}212p_2 + 0{\cdot}089w.$$

$$\Sigma \frac{H_{2D}}{sL} = \frac{103}{840}p_1 + \frac{3876}{18{,}256}p_2 + \frac{29w}{326} = 0{\cdot}1226p_1 + 0{\cdot}212p_2 + 0{\cdot}089w.$$

The maximum bending moments due to the wind determine the design of this frame; the horizontal suction due to the wind on the roof and the leeward

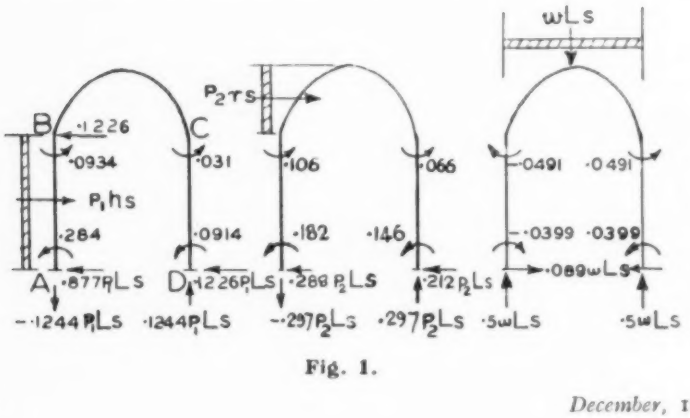

Fig. 1.

wall increases the moments due to the dead and imposed loads. The maximum moments on the columns occur at D and C. These moments are due to a suction of 7 lb. per square foot on the leeward wall and a pressure of 3 lb. per square foot on the windward wall, with a suction of 6·5 lb. per square foot and a pressure of 1 lb. per square foot on the leeward and windward vertical projections respectively of the roof.

Maximum Moments and Thrusts on the Columns.

$M_{DC} = 72{,}000[(7 \times 0{\cdot}284) + (3 \times 0{\cdot}0914) + (6{\cdot}5 \times 0{\cdot}182) + (1 \times 0{\cdot}146) + (30 \times 0{\cdot}0399)]$ in.-lb. = 344,750 in.-lb.

Vertical thrust on the centre line at D
$= (10 \times 0{\cdot}1244 \times 300) + (7{\cdot}5 \times 0{\cdot}297 \times 300) + (30 \times 150) + (14 \times 300) = 9740$ lb.

Horizontal thrust at D
$= 300[(7 \times 0{\cdot}877) + (3 \times 0{\cdot}1226) + (6{\cdot}5 \times 0{\cdot}288) + (0{\cdot}212) + (0{\cdot}089 \times 30)]$ lb.
$= 3314$ lb.

$M_{CD} = 72{,}000[(7 \times 0{\cdot}0934) + (3 \times 0{\cdot}031) + (6{\cdot}5 \times 0{\cdot}106) + (1 \times 0{\cdot}066) + (30 \times 0{\cdot}0491)] = 214{,}456$ in.-lb.

Vertical thrust at C = 9740 − 4200 = 5540 lb.

Horizontal thrust at C
$= 300[(6{\cdot}5 \times 0{\cdot}288) - (4 \times 0{\cdot}1226) + (1 \times 0{\cdot}212) + (30 \times 0{\cdot}089)]$ lb. = 1279 lb.

Design of the Columns.

Assume that the effective height of a column is the height from the feet of the columns to half the rise of the arch. This assumption is made as the rib, the roof covering, the curved sheeting, and the wind bracing all contribute to the partial fixity of the top of a column. The side rails restrain the columns in the direction of the length of the building. Therefore for a column 12 in. by 10 in. with an effective height of 25 ft., $\frac{h}{D} = \frac{300}{12} = 25$ and the reduction-coefficient for the stress in a long column is 0·677. B.S. Code No. 114 allows an increase of $33\frac{1}{3}$ per cent. in the allowable stresses for the effects of wind forces, so that the reduction-coefficient may be increased to 0·888. The allowable stresses are then $t = 0{\cdot}888 \times 18{,}000 = 16{,}000$ lb. per square inch and $c = 0{\cdot}888 \times 1100 = 980$ lb. per square inch. Paragraph 312(*c*) of the Code allows the permissible stresses to be determined without reference to the reduction-coefficient for sections within a distance of $\frac{L}{8}$ from the base of the column. Let x_1 be the distance of zero moment (point of contraflexure) from D, and the suction due to wind on column DC = 7 × 15 = 105 lb. per foot of height; then the equation for the moments on the column $= \frac{344{,}750}{12} - 3314x + \frac{105x^2}{2} = 0$, and $x_1 \simeq 10$ ft. from D.

At a distance of $\frac{L}{8}$ from D(= 2·5 ft.), $M = 249{,}268$ in.-lb. and the vertical thrust = 8990 lb.; therefore $e = 27{\cdot}8$ in. Using two bars, 1-in. diameter, on both the tensile and compressive faces of the column, with 1 in. of cover, $c = 815$ lb. per square inch and $t = 14{,}300$ lb. per square inch.

At the base of the column, $M_{DC} = 344{,}750$ in.-lb., the vertical thrust = 9740 lb., $e = 35{\cdot}4$ in., $\frac{e}{d} = 3{\cdot}37$, and with the same reinforcement $c = 1120$ lb.

per square inch and $t = 20{,}800$ lb. per square inch. The allowable stresses due to the effects of wind are $t = 24{,}000$ lb. per square inch and $c = 1467$ lb. per square inch. At a distance of 4 ft. from D the bending moment is 195,758 in.-lb. so that the four 1-in. diameter bars need extend only 4 ft. from the base and be lapped for a distance of 3 ft. with the main reinforcement of the column which is designed to resist the moment and thrust at the top of the column.

At C the moment $M_{CD} = 214{,}456$ in.-lb., the vertical thrust = 5540 lb., and $e = 38{\cdot}8$ in. Using four bars of $\frac{7}{8}$-in. diameter at the corners of the column, $c = 755$ lb. per square inch and $t = 16{,}700$ lb. per square inch. The stress in the steel is only slightly greater than the allowable stress of 16,000 lb. per square inch, and hardly merits an increase in the area of reinforcement, but four $\frac{1}{2}$-in. bars may be lapped with the $\frac{7}{8}$-in. bars at this section.

Design of the Rib.

The stresses in the rib will be calculated at two points, namely, at half the rise ($y = 5$ ft.) and at the crown ($y = r = 10$ ft.). The critical moments are those due to the suction of the wind and the vertical load giving the greatest value of M_{CD}. Transferring the origin of the equation of the centre-line of the rib to any point at a distance x ft. from C, the equation becomes

$$y = 2x - \frac{2x^2}{L} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

By taking moments about any point on the centre-line of the arch at a distance x ft. from C, the moment at any section of the arch is

(*a*) Due to the load p_1:

$$pL^2s\left[\left(0{\cdot}1244\frac{x}{L}\right) + \left(0{\cdot}4 \times 0{\cdot}1226\frac{y}{L}\right) - (0{\cdot}0934 \times 0{\cdot}7) - (0{\cdot}031 \times 0{\cdot}3)\right].$$

(*b*) Due to the load p_2:

$$pL^2s\left[\left(0{\cdot}75 \times 0{\cdot}297\frac{x}{L}\right) - \left(0{\cdot}65 \times 0{\cdot}288\frac{y}{L}\right) - (0{\cdot}1 \times 0{\cdot}212y) - (0{\cdot}066 \times 0{\cdot}1) + \left(\frac{0{\cdot}65y^2}{2L^2}\right) - (0{\cdot}106 \times 0{\cdot}65)\right].$$

(*c*) Due to the load $w = 3p$: $pL^2s\left(1{\cdot}5\frac{x}{L} - \frac{3x^2}{2L^2} - 0{\cdot}267\frac{y}{L} - 0{\cdot}1473\right)$.

By adding these equations and substituting $p = 10$ lb., $s = 15$ ft., and $L = 20$ ft., the expression for the moment may be written

$$M_x = 0{\cdot}4875x^4 - 19{\cdot}5x^3 + 97{\cdot}908x^2 + 2982{\cdot}09x - 17{,}848{\cdot}8.$$

If $x = 6$ ft., the moment at this point is 11·8 ft.-lb. and therefore the point of contraflexure is slightly less than 6 ft. from C.

At half the rise of the arch $y = 5$ ft., $x = 2{\cdot}93$ ft., and the moment is $-8408{\cdot}7$ ft.-lb., $H = 791$ lb., and V = vertical load = 4223 lb. To obtain the direct thrust on the centre-line of the rib, multiply the resultant of H and V by the cosine of the angle between this resultant and the slope of the centre-line of the arch at the point $x = 2{\cdot}93$ ft. and $y = 5$ ft. Thus, from equation (2),

$$\frac{dy}{dx} = \tan\theta = 2 - \frac{4x}{L}; \text{ for } x = 2{\cdot}93, \tan\theta = 1{\cdot}414,$$

and the slope of the arch at this point is 54 deg. 44 min. The resultant is $\sqrt{4223^2 + 791^2} = 4290$ lb. acting at an angle whose tangent is $\frac{4290}{791}$, that is 79 deg. 47 min. The direct thrust is therefore

$$4290 \cos (79 \text{ deg. } 47 \text{ min.} - 54 \text{ deg. } 44 \text{ min.}) = 3880 \text{ lb.}$$

The depth of the rib at this section is 11·44 in., $M = 100{,}904$ in.-lb., $e = 26{\cdot}1$ in., and, using two $\frac{3}{4}$-in. bars at the top and bottom of the rib, $c = 487$ lb. per square inch and $t = 11{,}200$ lb. per square inch.

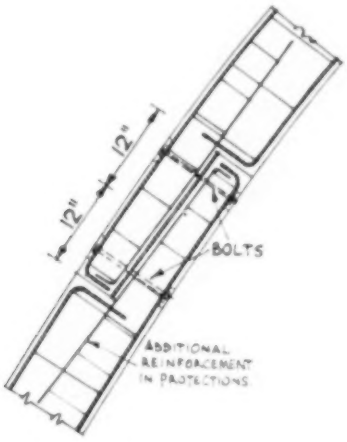

Fig. 2.

At the crown, $y = 10$ ft., $x = 10$ ft., $M = 7146{\cdot}7$ ft.-lb., and $H = 304$ lb. Considering only the vertical load, $M = 5652$ ft.-lb. and $H = 801$ lb. Where $M = 85{,}760$ in.-lb., $e = 282$ in., $d = 8{\cdot}2$ in., and $\frac{e}{d} = 34{\cdot}4$. With two $\frac{3}{4}$-in. bars at the top and bottom the percentage of steel is 2·17, and, for equal areas of steel at the top and bottom, the limiting value of n_1 for bending combined with a small thrust is 0·373. For this value of n_1, $c = 537$ lb. per square inch and $t = 13{,}350$ lb. per square inch. For $M = 67{,}824$ in.-lb., $e = 85$ in., a percentage of steel of 2·18, and $n_1 = 0{\cdot}385$, $c = 438$ lb. per square inch and $t = 10{,}500$ lb. per square inch.

Thus the reinforcement in the rib from crown to half the rise is two $\frac{3}{4}$-in. bars, in the top and bottom, and from half-rise to springing two $\frac{7}{8}$-in. bars, in the top and bottom, with 1 in. of cover. Lateral ties, $\frac{1}{4}$ in. diameter at 9-in. centres, must be provided to restrain the bars.

The main reinforcement of the columns comprises four $\frac{7}{8}$-in. bars, with four $\frac{1}{2}$-in. bars extending 2 ft. 6 in. on each side of the junction with the arch rib, the $\frac{7}{8}$-in. bars being continuous with the bars in the arch at the junction. At the base of the column four 1-in. bars, securely anchored in the base by hooks

or bends, should extend 4 ft. above the base and lap 3 ft. with the $\frac{7}{8}$-in. bars in the column.

The construction of the frame may be made more simple by precasting it in three parts, namely, the two columns with the parts of the rib projecting to the point of contraflexure, and the remainder of the rib. The joint between these portions will be scarfed as shown in *Fig.* 2, the joint extending for about 12 in. on each side of the point of contraflexure and bolted at about 4 in. from the ends of the joint. It is not necessary to increase the dimensions of the rib at the joint, but as the projections act as cantilevers they must be suitably reinforced. It is more convenient for erection for the joint to be as shown with the projection from the rib forming the upper portion of the joint rather than to have the projection from the rib at the bottom. This has also the advantage that under vertical loads the inside faces of the joints are in compression.

To secure adequate fixity of the columns the base should be large enough (depending on the ground pressure) to prevent rotation. The best method appears to be to place the column eccentrically on the base so that under the effects of the bending moments and thrusts due to the dead and live loads (but not the wind) the ground pressure is uniformly distributed. For this frame the outward bending moment due to the vertical load is $(0{\cdot}0399wL^2 + 0{\cdot}89Lwsd_1)$, where d_1 is the depth of the base. Thus for the forces due to the vertical load V_1 at the column base (the weight W_1 of the base may be ignored in the calculations except for small allowable ground pressures), and, calculating moments about the centre-line of the underside of the column base, $V_1e = 0{\cdot}0399wL^2s + 0{\cdot}089wLsd_1$, so that $e = \frac{wLs}{V_1}(0{\cdot}0399L + 0{\cdot}089d_1)$. The base is then to be designed taking into account the bending moment due to the wind so that the allowable ground pressure is not exceeded. If it is considered necessary to provide greater fixity against rotation or flexibility, the base can be made deeper or deeper and wider depending on the nature of the ground.

Compacting Sand.

A MACHINE, known as a "Vibroflot," is being used to consolidate sand on the site of a large chemical works in Florida. According to "Mechanical Engineering" for February, 1953, the process will save about £90,000 on this site and six months after erection the building has not settled on the sand although theoretically it should have settled 1 in. By means of a vibrated tube, columns of compacted sand of about 8 ft. to 10 ft. diameter are formed, and it is claimed that these columns strengthen the whole of the sand under a building and greatly increase its resistance to pressure.

The tube is vibrated by an electrically driven eccentric which produces a centrifugal force of ten tons. The tube is suspended from a crane, and as it is lowered into the sand (while vibrating) a jet of water is forced through its tip so that the tube sinks through saturated sand. On the site at Florida the tube was sunk to an average depth of 13 ft. As the tube sinks, sand is shovelled in to fill the space formed by the compaction of the surrounding soil. The tube is withdrawn in stages of 1 ft., and vibrated at each 1 ft. of depth until it is thought that maximum compaction of the surrounding sand has been obtained. An average of 2½ cu. yd. of sand is used to fill the space made by the vibrator. The vibrating tube is used at about 8 ft. centres so that the compressions caused by the bores overlap (the diameter of the tube is not stated). The machine is being used by the Rust Engineering Company, of Pittsburgh, U.S.A.

Accelerator and Cyclotron Building at a London Hospital.

THE new research building at Hammersmith Hospital, London, for the Medical Research Council, consists of three sections each separated from the others by expansion joints to allow for unequal settlements of the structures due to differences of the loads and types of construction (*Fig.* 1).

The central building at present has four stories each 11 ft. high, but it is intended to add a further four stories later. The building is a framed structure 120 ft. long and 36 ft. wide with a row of columns longitudinally to provide spans of 14 ft. 6 in. and 21 ft. 6 in. The floors, in which heating coils are embedded, are 6 in. thick plus finishes 1½ in. thick. The present building contains workshops, laboratories, etc., and the future floors will be used mainly for administrative purposes.

At one end of the central structure is the linear accelerator building and at the other end is the cyclotron building. These structures are designed to give full protection against the escape of radiations, and the walls and roofs have a minimum thickness of 6 ft. of very dense concrete. A small amount of reinforcement was used and 1 : 2 : 4 concrete placed and compacted by immersion vibrators. It was necessary to ensure that there should be no straight joints; no vertical joints are used, and horizontal joints are in grooves 6 in. deep by 6 in. wide. The extract ventilation pipes passing through the wall of the cyclotron chamber have right-angle bends to prevent the escape of radiations (*Fig.* 2).

Fig. 1.

Fig. 2.—Air-Pipes in the Walls of the Cyclotron Chamber.

The linear accelerator building has a treatment room about 14 ft. 6 in. square by 17 ft. high. An electrically-operated floor can be altered in level so as to allow the operator to adjust the position of the floor rather than that of the accelerator which is very accurately fixed in position.

The cyclotron building houses a cyclotron weighing about 110 tons which had to be very accurately erected and its position relative to the structure maintained. The chamber is 20 ft. 6 in. by 44 ft. 6 in. by 16 ft. 6 in. high, with access to an assembly bay which is blocked, when the cyclotron is in use, by a massive sliding door (operated by a ¾-h.p. motor) to afford protection to the staff. This door is about 22 ft. 6 in. long, 14 ft. high, and 10 ft. thick; it weighs about 185 tons and is supported by a framework of rolled steel joists carried on roller bearings. The mechanism for operating the door was designed and made by Messrs. F. Piper & Sons, Ltd. The door is constructed of reinforced concrete, using precast concrete blocks 9 in. thick as external shuttering; this was considered to be the only method by which sufficient accuracy could be achieved to allow for the specified clearances of ¼ in. between the door and the surrounding concrete. The base for this door is shown in *Fig.* 3.

A similar type of door, but of smaller size, seals the entrance from the adjoining

waiting room. In this case a steel box was mounted on roller bearings as in the case of the larger door and then filled with concrete (designed by Colcrete, Ltd.) weighing over 300 lb. per cubic foot. The thickness of the concrete inside the steel box is 3 ft. and the door weighs about 35 tons. This concrete has the same resistance to the penetration of rays as the thicker concrete wall surrounding it.

All the electrical apparatus, etc., required to operate the cyclotron magnet is in a lightly-reinforced concrete-framed two-story building, adjacent to the chamber, which also contains several laboratories. On the roof of these two buildings is the cooling-plant house, constructed of reinforced concrete walls 6 in. thick with semi-circular ends.

The contractors for the whole of this work were Sir Robert McAlpine & Sons, Ltd. The architect was Professor Basil Ward, F.R.I.B.A., of Messrs. Ramsey, Murray & White, and the structural engineers were Considère Constructions, Ltd. The photographs are reproduced by courtesy of the Medical Research Council.

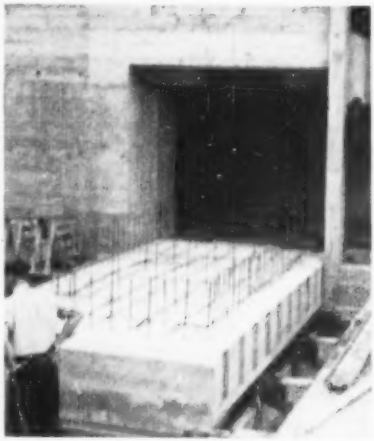

Fig. 3.—Base for the 10-ft. thick Door.

A Prestressed Gas-Holder Tank.

A PRESTRESSED gas-holder tank recently completed at Cromer for the Norwich Division of the Eastern Gas Board is believed to be the first of its kind to be built in this country. The tank (*Fig.* 1) has an internal diameter of 74 ft. and a depth of 27 ft. 6 in. The wall is constructed of gunite and is prestressed circumferentially by 0·2-in. high-tensile cold-drawn wire which is wound continuously around the wall by the equipment shown in *Fig.* 1. The wire passes from spools on the suspended carriage through a former which reduces the diameter to 0·177 in., and the force required to pull the former round the wall tensions the wire. The wall of the tank is stressed by a single layer of wire and the heavy ring around the top by two layers of wire with gunite placed between the layers. The wall is prestressed vertically by ¾-in. high-tensile bars at 3 ft. 3 in. centres. The total amount of wire used was about 6 tons.

The main contractors were Messrs. Newton Chambers & Co., Ltd., and the contractors for the prestressing were Preload (Great Britain), Ltd.

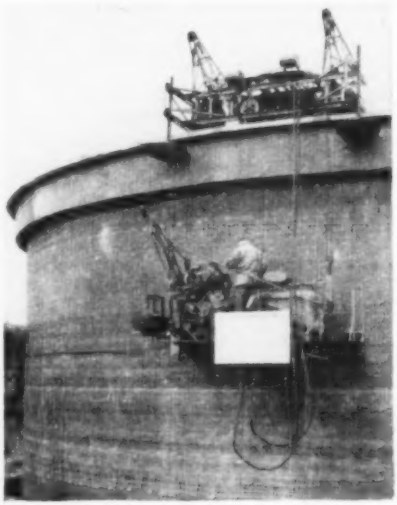

Fig. 1.—Winding Prestressing Wire around Wall of Tank.

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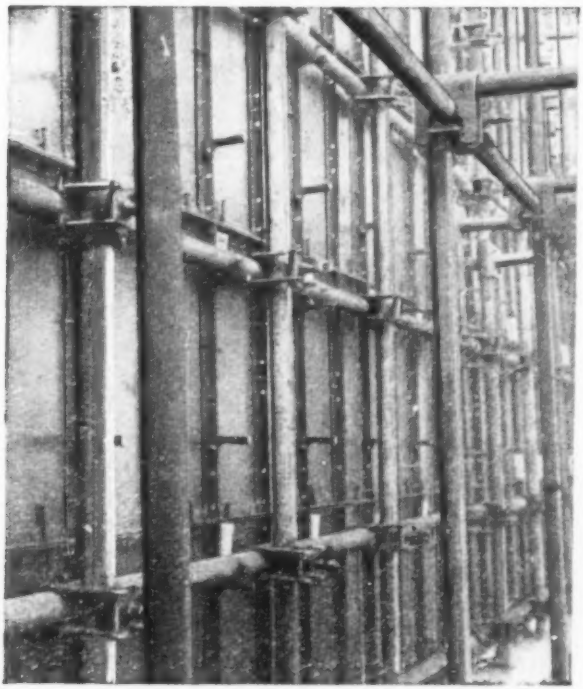

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Research on Prestressed Roads.

THE following is abstracted from the report for the year 1952 of the Road Research Board (H.M. Stationery Office. Price 4*s.* 6*d.*).

Friction due to Formation.

The movements of slabs have been measured on prestressed roads at Crawley New Town and in Hampshire. The amount of prestress in a slab depends upon the freedom with which the slab can contract under the effect of the prestressing forces. Some resistance to free movement is imposed by the formation and it is necessary to know what this resistance is in order to determine how much residual stress remains in the concrete. At Crawley the slab has begun to slide slightly more freely than when constructed. A slab 40 ft. long by 10 ft. wide by 6 in. thick laid on waterproof paper on sand was pulled (*Fig.* 1) through distances up to 3 ft. and the restraint due to the formation was determined for different movements (*Fig.* 2). It was found that (1) The force to start movement was 84 per cent. greater than that to maintain or re-start movement; (2) All the forces were considerably less than the normal prestressing forces for such a slab; (3) When movement had started the formation restraint was about 0·5; and (4) Formation restraint decreased only slightly with further movement.

Precautions to be taken in the construction of prestressed roads have been recommended, including stronger concrete, transverse reinforcement near the anchorages, protection against flying wedges, distant-reading devices for observing the extension applied to the cables, and goggles to protect the eyes against flying fragments of concrete. The necessity for greater precaution in road work arises because the slabs being stressed are thinner and longer than in normal construction. Because the slabs are thinner there are greater stresses at the anchors and therefore greater chance of failure, and longer slabs mean that more energy is stored in each cable and therefore greater damage is likely when failure occurs.

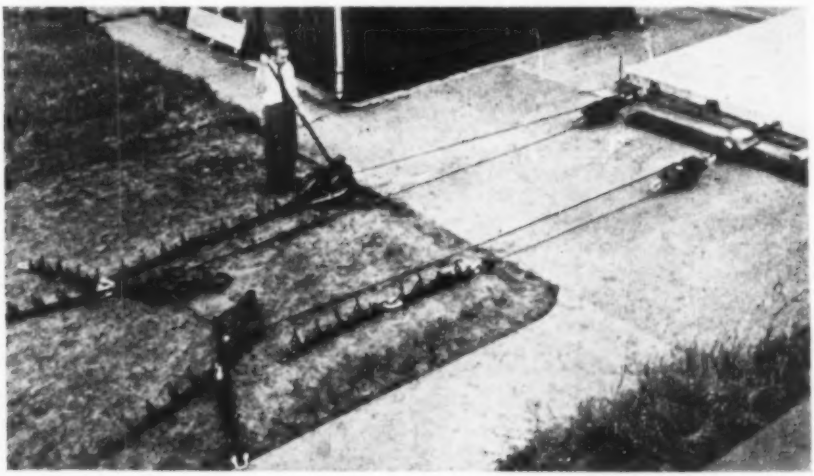

Fig. 1.—Measuring Restraint of Prestressed Slab.

Distribution of Stresses.

Tests have been made to determine the distribution of stresses due to the application of a force on the edge of a slab. The strain-gauge used consisted of four wire-resistance strain-gauges cemented to

a mild-steel plate having enlarged ends, the central portion of the gauge being protected by a brass cover so that the concrete exerted force on the enlarged ends only. The gauges were connected as a Wheatstone bridge, and the strains determined by measuring the out-of-balance voltage on a potentiometer. Several such gauges were cast along the main axis of a rectangular slab 6 ft. long by 1 ft. 3 in. wide by 6 in. deep. The gauges were parallel to the 1-ft. 3-in. side and the slab was subjected to a pair of line loads which were directly opposite each other and normal to the 6-ft. by 6-in. faces. This simulated the force applied by a single prestressing cable. By shifting the application of the load along the slab, and reading every gauge for each position of the loads, the distribution of strain within the slab was obtained. In *Fig.* 3 the experimental results are plotted together with the results of a theoretical investigation and those of a photo-elastic test. The slab was also subjected to two pairs of line loads $3\frac{1}{2}$ in. apart. The strains measured by the gauges were identical with values

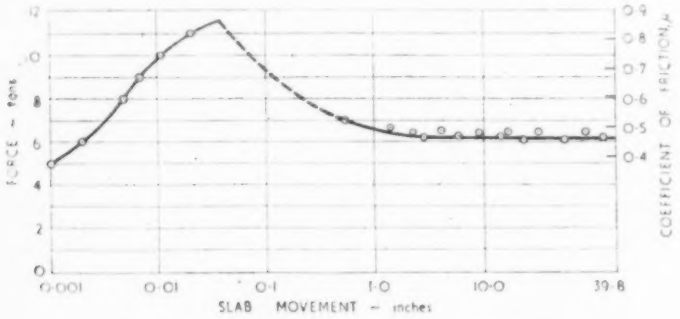

Fig. 2.—Formation Restraint on a Slab.

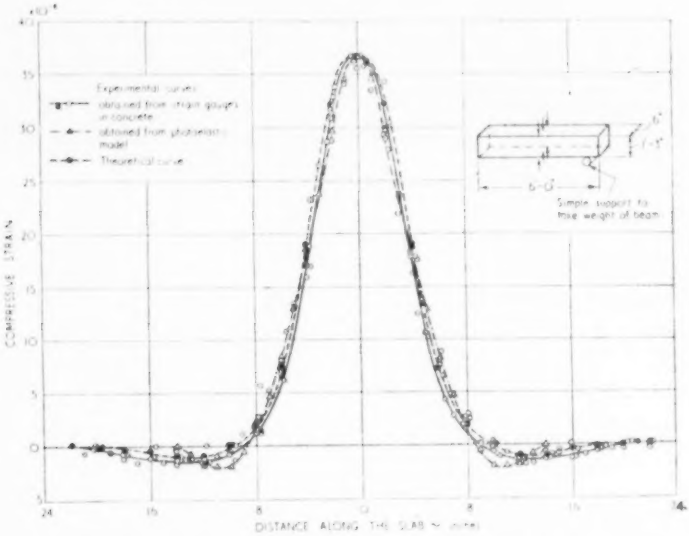

Fig. 3.—Distribution of Strain in a Prestressed Slab subjected to a Single Pair of Collinear Concentrated Loads.

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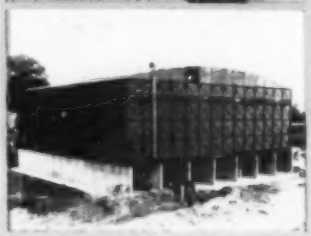

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given by the algebraic summation of the strains produced by each pair of loads. The distribution of strain was computed for pairs of loads at various spacings, and the most uniform distribution occurred when the loads were separated by a distance 0·4 times the width of the slab. When eight pairs of concentrated loads spaced at 6-in. intervals were applied to the 6-ft. by 6-in. sides, the stress varied only from 199 to 212 lb. per square inch between the second and seventh pair of loads. The conditions required for uniform distribution of stress vary with the size and shape of the specimen and with the method of loading. These conclusions apply to a small slab loaded at its sides and not necessarily to a very long slab, as in a road, loaded at its ends.

To investigate the distribution of strain near a concentrated load, a slab 5 ft. square by 6 in. thick was cast with 27 gauges at various points. Two loads of 10 tons were applied, as shown in *Fig.* 4, by two steel plates each 6 in. square, giving a loading similar to that produced by one of the methods of post-tensioning. The strains measured at depths of 6 in., 1 ft. 6 in., and 2 ft. 6 in. below the point of loading are shown in *Fig.* 4. The

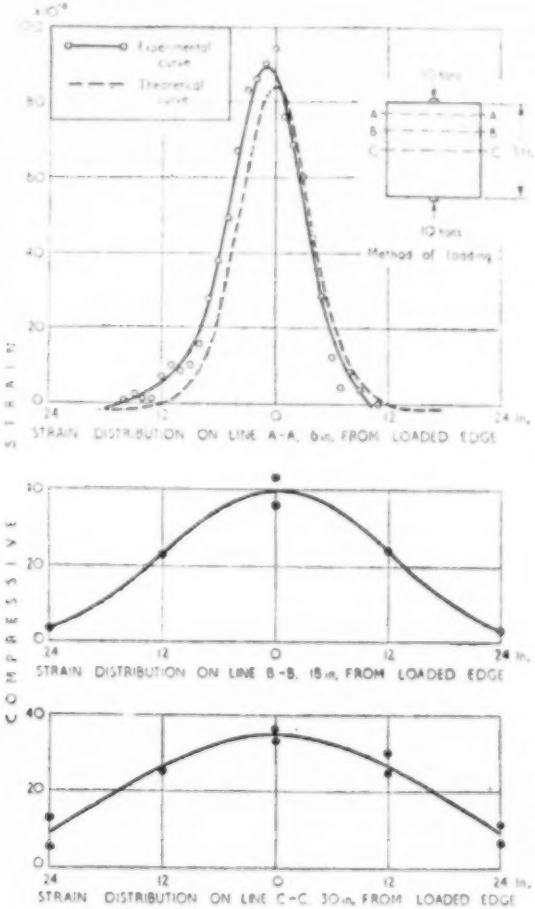

Fig. 4.—Distribution of Strain in a Prestressed Slab 5 ft. Square.

distribution of strain 6 in. from the loaded edge of the slab has been computed for a semi-infinite slab and this is shown in *Fig.* 4. Gauges cast in a slab containing holes formed by inflatable rubber tubes showed that the distribution of strain was not affected by the presence of the holes.

Prestressed Beams of Steel and Concrete.

A METHOD of prestressing beams consisting of rolled steel joists encased in concrete, called " Preflex " beams, is described by MM. Louis Baes and A. Lipski in the Belgian journal " Précontrainte-Prestressing " for January–June, 1953. The method is being used in the construction of civil engineering works and buildings in Belgium and the Belgian Congo. The principle of the method is that a rolled steel joist is bent so that it has a deflection about equal to, and in the same direction as, the deflection which will be caused by the imposed loads. While the beam is held in the bent position concrete is cast around the bottom flange, and when the concrete has hardened the beam is freed and its elastic recovery compresses the concrete. It is claimed that the method allows a reduction in the depth of construction, and, as the concrete will not in general be subjected to tensile stresses, it will protect the steel more effectively than a non-prestressed encasement which would crack due to deflection of the beam under load.

The joist is bent by means of jacks until the maximum tensile stress in the steel is equal to the maximum tensile stress in the encased beam under the total load; usually this is about 90 per cent. of the total load. Before the beam is bent it may be cambered so that there will be no deflection under working load. High-tensile steel with a yield stress of about 23 tons per square inch and a working stress of about 16 tons per square inch is used. The concrete has a crushing strength on 8-in. cubes of at least 4250 lb. per square inch at the time when it is prestressed.

When the beams are used in a concrete floor or roof they are designed as composite tee-beams (*Fig.* 1). A method of obtaining rapidly the approximate size of

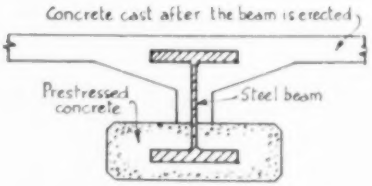

Fig. 1. The " Preflex " Beam.

the beams, and an approximate method and a precise method of calculating the stresses, are given. Generally the cross-sectional area of concrete is about ten times the cross-sectional area of the bottom flange. This concrete, in conjunction with the slab cast after the beam is in position, results in a beam which may be assumed to have a modulus of section about 10 per cent. greater than that of the steel beam only. Similarly the deflection of the composite beam compared with that of the steel beam only is about 1/2·5. Therefore the approximate size of the steel beam may be obtained from

$$Z \geq \frac{1}{1{\cdot}1}\frac{M}{f_t},$$

and

$$I \geq \frac{1}{2{\cdot}5}\cdot\frac{5}{384}\cdot\frac{qdl^3}{E_s\left(\frac{f}{l}\right)_q}$$

where M is the maximum bending moment, f_t the allowable stress in the steel beam, q the uniformly-distributed imposed load, d the spacing of the beams, l the span, E_s Young's modulus for the steel, and $\left(\frac{f}{l}\right)_q$ the ratio of the allowable deflection under the imposed load to the span. Having determined the approximate size of the beam, the stresses are checked by one of the methods described.

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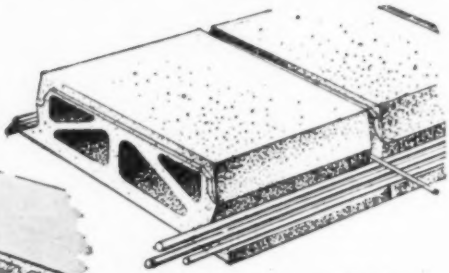

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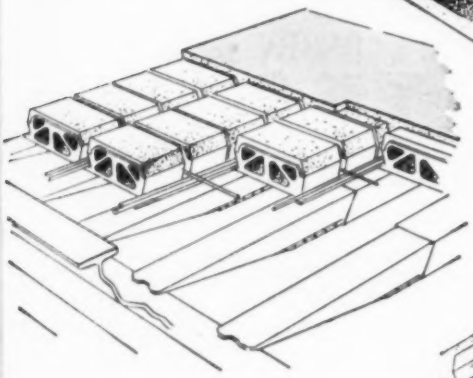

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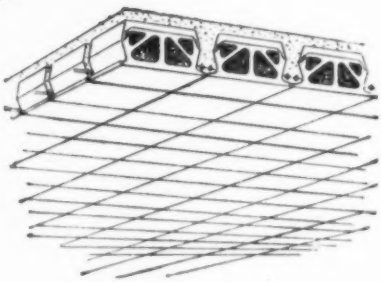

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(Continued on page 404.)

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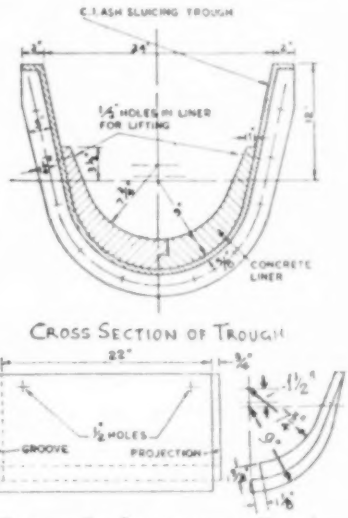

expensive and difficult compared with replacement of the concrete linings now installed which, because of their greater thickness at the base, have a much longer life.

The linings were cast in halves in wooden moulds in lengths of 1 ft. 10 in.; each half weighs 57½ lb. The foregoing is abstracted from the "Indian Concrete Journal" for July, 1953.

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MISCELLANEOUS ADVERTISEMENTS.

(*Continued from page 403.*)

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(*Continued in Col. 2.*)

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THE International Union of Testing and Research Laboratories for Materials and Structures has arranged a symposium on the non-destructive testing of concrete to be held in Paris between January 11 and 17, 1954. Further information may be had from M. Robert l'Hermite, Director of the Laboratoires du Bâtiment et des Travaux Publics, 12 rue Brancion, Paris XVe, France.

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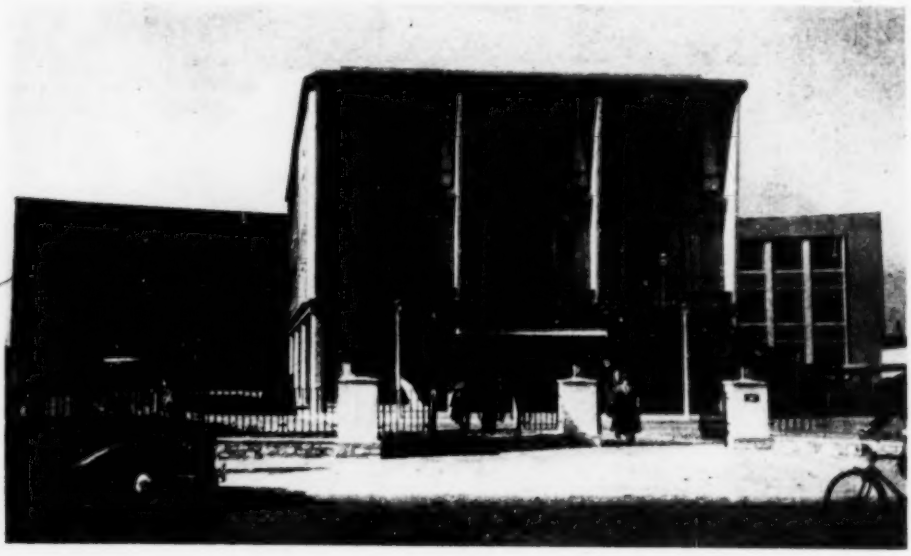

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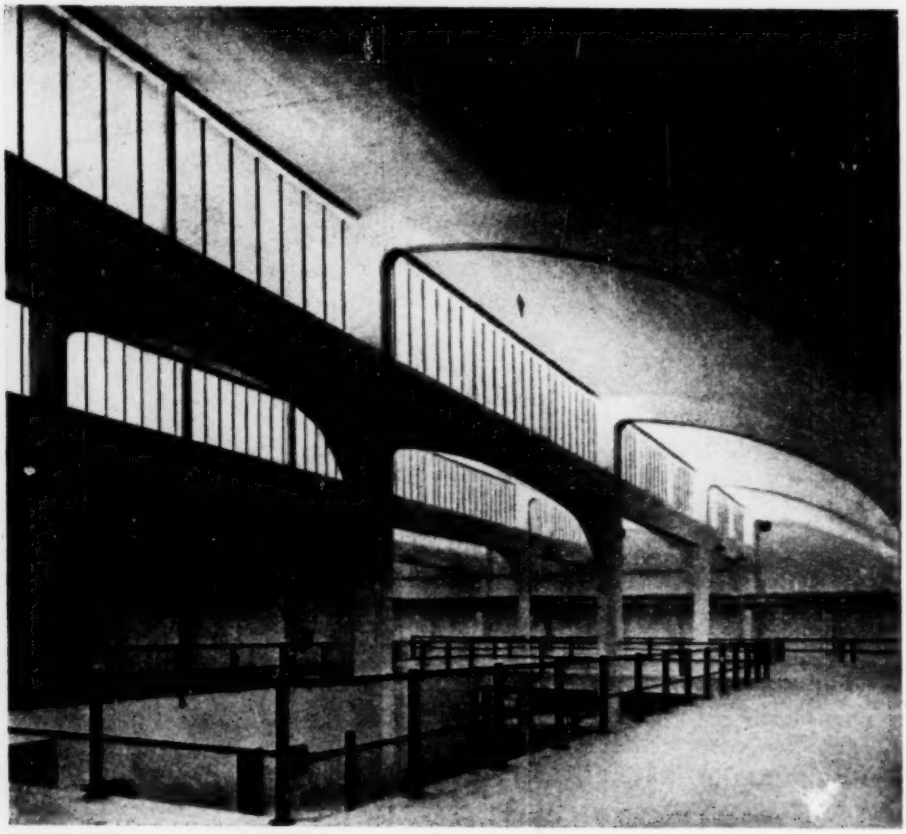

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